Eastside HCT Corridor
I-90 Floating Bridge (Homer Hadley) Expansion Joint
Final Conceptual Report

Prepared for:
Sound Transit

Prepared by:
Sound Transit East Link Project Team

January 2008

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Resource Lead: Ahmet Ozkan, P.E.. (AO) **Date Submitted for review:** (Date)

Senior Reviewer: Daniel P. Russell, P.E. (DPR) **Review completion date:** (Date)

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EAST LINK PROJECT

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SOUND TRANSIT EAST LINK PROJECT
Phase 2

I-90 Floating Bridge (Homer Hadley) Expansion Joint Final Conceptual Report

Prepared for:
CH2M HILL, Inc. and Sound Transit

Prepared by:
INCA Engineers, Inc.

January 2008

Translation services and information in accessible formats are available upon request by calling 1.800.201.4900 (voice) or 206.398.5410 (TTY).

For more information about the Expansion Joint Analysis for the Existing I-90 Floating Bridge Final Conceptual Report call (refer to specific community relations coordinator as appropriate) or write Sound Transit, 401 South Jackson Street, Seattle, WA 98104-2826. You may also e-mail Sound Transit at main@soundtransit.org, visit our Web site at www.soundtransit.org or call our toll free information line at 1-800-201-4900.

Certificate of Engineer

The work contained herein was prepared under the supervision and direction of the undersigned

EXPIRES 10/5/

Ahmet Ozkan
Chief Bridge Engineer

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Chapter 1

Executive Summary

As the Light Rail Vehicle crosses the I-90 (Homer Hadley) Floating Bridge, the vehicle must pass onto and off the transition spans. At each end of the transition spans, the vehicle must cross a joint that allows longitudinal expansion and contraction as well as horizontal and transverse rotation. This report defines the magnitude of the movement at the joint, the anticipated safe operating speed across the joint and further develops the joint details. This report concludes that:

- The WSDOT design criteria joint movements include both normal and extreme events. WSDOT provided the tabulated Homer Hadley expansion joint movements tabulations for annual and ultimate events on July 26, 2007. The Homer Hadley expansion joint movement information provided by WSDOT is included in Appendix A. Based on this expansion joint data the design team concluded that:
- Maximum Longitudinal Translation Movement can be significantly greater than the design criterion value of +/-24.5 inches under an ultimate event that includes Temperature, Longitudinal Breaking Force from Live Load, and Wind and Wave effects. Therefore, the Maximum Longitudinal Translation movement is limited by longitudinal restrainers at ± 18 inches. However, the design criterion Longitudinal Translation is specified as ± 24.5 inches.
- The Maximum Vertical Angle Change is shown as 0.8 degree for the annual event and 1.25 degrees for the ultimate event. However, the design criterion Vertical Angle Change is specified as 2.2 degrees.
- The Maximum Horizontal Angle Change is shown as 0.5 degree for the annual event and 1.0 degree for the ultimate event. However, the design criterion Horizontal Angle Change is specified as 1.1 degrees.
- Since detailed calculations were not available from WSDOT, the correlation between the joint movements and the initiating events have not been fully determined. However, the design team has identified joint movements for many of the significant loadings under normal operating conditions. It has been determined that the most severe vertical rotation is caused by lake level fluctuations. Joint curvatures and the resulting light rail vehicle operating speeds during these events have been calculated, and are:
  - For the average lake levels 20 feet or higher: 40 mph across joints
  - For occasional lake level drops under 20 feet: 30 mph across joints

  The only one time that the Lake Washington level has fallen below Elevation 20.0 feet during the service life of the Homer Hadley Floating Bridge to date was over a continuous 2-month period in the Fall of 1986.

  As the East Link Project advances into the preliminary and final design phases, the consultant team will re-evaluate the proposed operating speeds in more detail. If required, the models will be further refined to investigate the operational aspects of the expansion joints in order to insure the structural integrity and comfort of the riders.
- A 36-foot long three-beam transition concept can accommodate all required movements across the joints. These have been further analyzed and detailed.

Chapter 2

Introduction

2.1 I-90 Homer Hadley Floating Bridge

In the Phase-I of the ST East Link Project, the consultant team conducted a conceptual study and prepared a report titled *"I-90 Floating Bridge (Homer Hadley) Expansion Joint Study"*. In the report, examples of modern rail bridge expansion joints and their design criteria movements were compared with the anticipated design criteria movements of the I-90 Floating Bridge. Two such bridges and corresponding expansion joint concepts are the Tagus River Suspension Bridge (Telescoping Girder Concept), in Lisbon, Portugal; and the Sky Train Cable Stayed Bridge (Spring Support Concept), in Vancouver, Canada, both of which have a successful history of passenger rail operation. The report also included a previously developed design concept (Transition Beam Concept) for the rail joint by John Insco Williams, January 1986.

The report concluded that the Telescoping Girder Concept was not a good fit for the I-90 Bridge expansion joint due to its limited movement capacity. Therefore, only the Spring Support and Transition Beam concepts were studied further. After these conceptual evaluations, it was concluded that although either of the two concepts considered, spring support or transition beam concepts, could be made to work, the three-beam transition concept was much easier to implement.

The objective of this report is to undertake additional structural analyses to further develop the three-beam transition concept details to a 10 percent design level and develop quantities for cost estimating purposes. This report also includes a detailed discussion regarding the contributions of different load conditions on the maximum design joint movement criteria set by WSDOT. However, this discussion is limited to the information that was available to the consulting team during the time frame of this study.

Chapter 3

WSDOT Movement Criteria

3.1 Description

The maximum joint design movements for the expansion joints located between the fixed and floating spans of the bridge (at Pontoon Piers A and R) were obtained from the *1984 Homer Hadley Floating Bridge As-Built Plans* and are shown in Table 3-1. Table 3-1 also shows the joint movements that are listed in the KPFF report, *Homer Hadley Interstate 90 Floating Bridge – Draft Structural Feasibility Study: Light Rail Conversion, September 2001.* These later movements are larger than the original maximum joint movements listed in the 1984 As-Built Plans to account for the weight of the Light Rail Vehicle. The additional movements due to the Light Rail Vehicle were confirmed by the Load Test performed by WSDOT in 2006, and documented in the report, *Homer Hadley (Interstate 90) Floating Bridge Test Program for Light Rail Transit.*

TABLE 3-1
Joint Movements

Source	Longitudinal Translation X (in)	Vertical Rotation θv (deg)	Horizontal Rotation θh (deg)
1984 As-Built Plans	+/- 24.0	- 2.0	+/- 1.0
2001 KPFF Report	+/- 24.5	- 2.2	+/- 1.1

3.2 Observations on the WSDOT Movement Criteria

Third Lake Washington Bridge Design Criteria for Floating Structure by WSDOT, 1977 lists the Service Load Combinations as follows:

TABLE 3-2
Service Load Combinations

Loads	D	H	L	I	WN	NW	WS	SW	WL	LF	S	T	K	DM
Service Load Group														
S1 (100%)	1	1	1	1									1	
S3 (125%)	1	1	1	1	1	1			1	1			1	
S4 (125%)	1	1	1	1							1		1	
S6 (140%)	1	1	1	1	1	1			1	1	1	1	1	
S7 (140%)	1	1	1	1									1	1
S9 (150%)	1	1	1	1	1	1			1	1			1	1

Where,

D=	Dead Load (including anchor cable initial forces)
H=	Hydrostatic Pressure (at still water draft)
L=	Live Load (Highway or Rapid Transit Alternate)
I=	Live Load Impact
WN=	Normal Wind on Structure – 1 year storm
NW=	Normal Wave – 1 year storm
WS=	Storm Wind on Structure – 100 year storm
SW=	Storm Wave – 100 year storm
WL=	Wind on Live Load
LF=	Longitudinal Force from Live Load
S=	Shrinkage and Creep
T=	Temperature
K=	Change in Lake Level
DM=	Potential Damage.

Some observations on the WSDOT Design Criteria are as follows.

3.2.1 Relatively Minor Loadings

Among the loads listed above, D (Dead Load), H (Hydrostatic Pressure), WL (Wind on Live Load), and S (Shrinkage and Creep) have negligible or no impact on expansion joint movements.

3.2.2 Longitudinal Loading Effects

The loading LF (Longitudinal Force from Live Load) and T (Temperature) contribute only to longitudinal translation movement. One exception to this is the differential temperature between the top deck and the shaded part of the superstructures or submerged parts of the pontoons. In this case, the differential temperature will create some vertical rotations in addition to corresponding longitudinal translations. However, these vertical rotations are considered negligible.

3.2.3 Vertical Loading Effects

The load K (Change in Lake Level) is the main loading that produces vertical rotation. *Third Lake Washington Bridge Design Criteria for Floating Structure by WSDOT, 1977* lists the changes in lake level as:

- Maximum Rise: 0.8 feet
- Maximum Fall: 3.8 feet

These changes in lake level can be translated to the movements listed in Table 3-3.

TABLE 3-3
Lake Washington Level Movements

Lake Level Change	Longitudinal Translation X (in)	Vertical Rotation θv (deg)	Horizontal Rotation θh (deg)
0.8 ft Rise	+ 0.44	+ 0.245	0
3.8ft Fall	- 2.07	- 1.164	0

The Homer Hadley Floating Bridge "as built" drawings show the Lake Washington normal water level elevation as 8.02 feet based on City of Seattle Datum. A comparison to other Lake Washington Datum Planes, (Figure 3-1) shows the Elevation 8.02 feet based on the City of Seattle Datum is equivalent to 20.97 feet (8.02+6.13+3.57+2.35+0.90 = 20.97 feet) of the Corps of Engineers Datum. Under normal operating conditions, the Corps maintains Lake Washington within a 2-foot range between Elevations 20.0 and 22.0 feet. The design criteria lake level rise of 0.8 feet is slightly less than the Corps' normal lake level rise of (22.00-20.97=) 1.03 feet, and the design criteria lake level drop of 3.8 feet is considerably greater than the Corps normal lake level fall of (20.97-20.00=) 0.97 feet. From our brief discussion with Patrick Clarke of WSDOT, we understand that the 3.8 feet lake level drop accounts for certain extreme events, such as a sudden pool loss after an earthquake or a failed lock gate. Light Rail Operations across the joint will need to consider the fact that much of the 3.8-foot elevation fall is the result of extreme events.

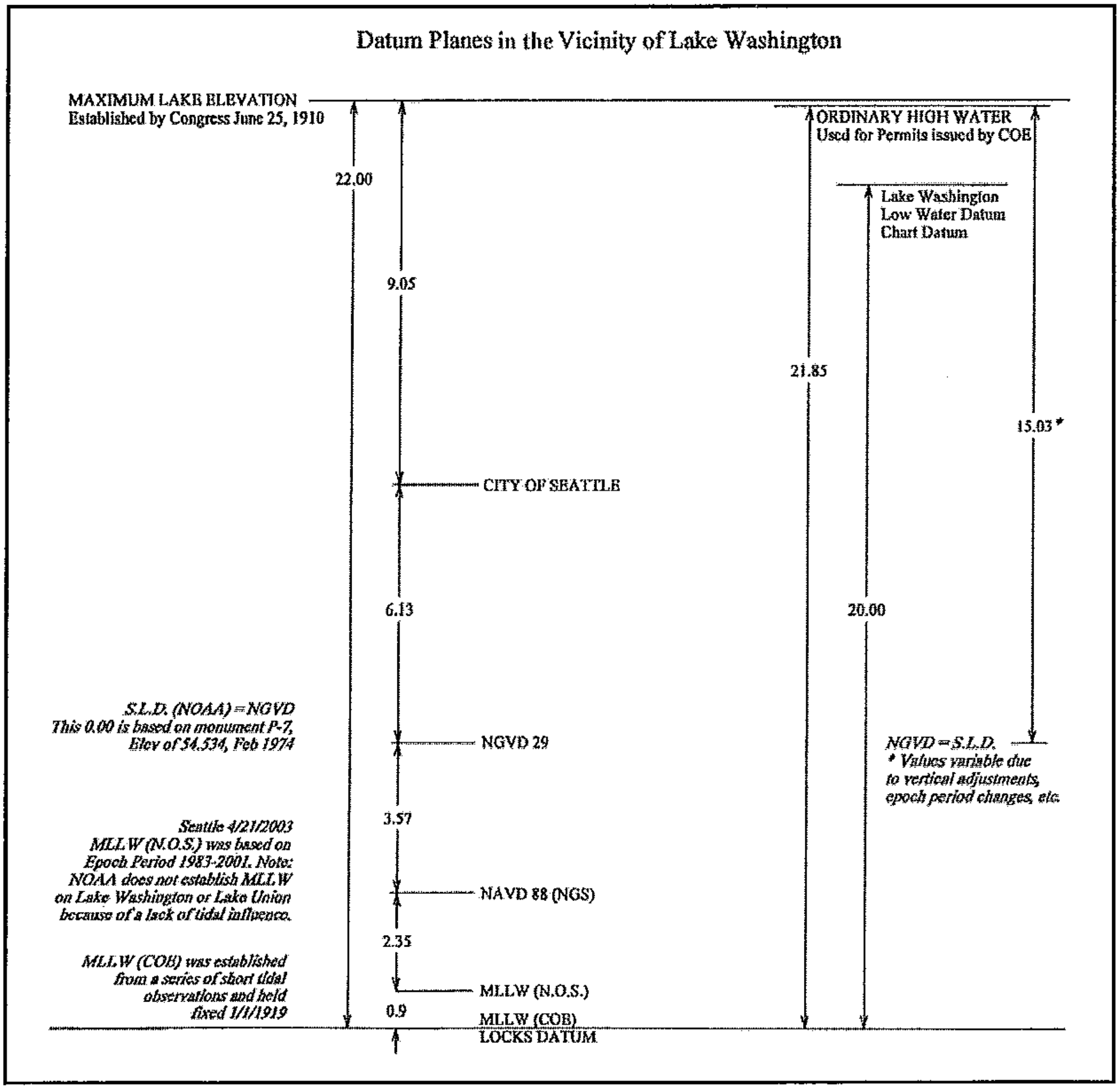


FIGURE 3-1
Lake Washington Datum Planes

Based on our communication with the Seattle District Corps' Hydraulic Engineer Lynne Melder, the minimum lake elevation is maintained during the winter months to allow for annual maintenance on in-take elements such as docks and walls, and to provide storage space for high inflow. This means that during the summer months when the lake level is kept high, relative movements of the transition joints (under normal operating conditions due to lake level change plus live load) will usually be near the minimum, as shown in Table 3-4. 3-4, which lists the Service Load 1 combination movements for the expansion joints at Pontoons A and R, and Piers 7 and 9. Due to lake level rise, the expansion joints at Pontoons A and R experience a crest curve, whereas at Piers 7 and 9, the expansion joint experiences a sag curve.

TABLE 3-4
Service Combination 1 Lake Washington Level High

Loading	Longitudinal Translation X (in)	Vertical Rotation θv (deg)	Horizontal Rotation θh (deg)
Live Load (L+1)			
HS25 + LRT	-1.01	-0.237	0.088
Lake Level Change (K)			
1.0 ft Rise	0.55	0.306	0
S1 Combination (L+I+K) *Crest Curve at Pontoons A & R*	-0.46	0.069 < θv < 0.306	0.088
S1 Combination (L+I+K) *Sag Curve at Piers 7 & 9*	-1.56	0.306 > θv > 0.069	0.088

On the other hand, during the winter months when the lake level is kept low, relative movements of the transition joints (under normal operating conditions due to lake level change plus live load) will be near the maximum as shown in Table 3-5. In that case, a sag curve occurs at Pontoons A and R and a crest curve occurs at Piers 7 and 9.

TABLE 3-5
Service Combination 1 Lake Washington Level Low

Loading	Longitudinal Translation X (in)	Vertical Rotation θv (deg)	Horizontal Rotation θh (deg)
Live Load (L+I)			
HS25 + LRT	-1.01	-0.237	0.088
Lake Level Change (K)			
1.0 ft Fall	- 0.55	-0.306	0
S1 Combination (L+I+K) *Sag Curve at Pontoons A & R*	-1.56	0.543 > θv > 0.306	0.088
S1 Combination (L+I+K) *Crest Curve at Piers 7 & 9*	-0.46	0.306 < θv < 0.543	0.088

However, in both cases, under normal operating conditions [Service Group 1 combination (S1)] the expansion joint movements will typically be much smaller than the design criteria movements.

3.2.4 Horizontal Loading Effects

Wind loads (WN, NW, WS, SW) and the potential damage (DM) loading are the main loads that produce horizontal rotation. It is our understanding that the potential damage (DM) could include extreme events such as a sudden anchor cable break. However, the consultant team has no information about possible lateral movements due to anchor cable break. The Transition Pontoons (Pontoons A and R) have two anchor cables each at the north and south sides. WSDOT design criteria have provisions for "severing of any one anchor line". It is not clear if the loss of one cable at Pontoon A or R will produce the enough lateral translation to create the design horizontal rotation value. The Table 3-6 includes the movements due to wind loads as shown in the *"Wave Loading Analysis of Lake Washington Bridges and Results, New I-90 Floating Bridge, May 1983,"* by Glosten Associates. The Glosten report includes five degrees of freedom motion response values (sway, heave, roll, pitch, and yaw). The roll, pitch, and yaw contributions to the movements shown in Table 3-6 are negligible and not thus included in the movement calculations. The 1-year and 100-year storm movements include only the sway and heave components.

TABLE 3-6
1-Year and 100-Year Storm Events

Loading	Longitudinal Translation X (in)	Vertical Rotation θv (deg)	Horizontal Rotation θh (deg)
1 Year Storm	0.17	0.018	0.033
100 Year Storm	0.59	0.050	0.115

Table 3-1 indicates the maximum joint movements for which the joints were designed. Comparing Table 3-6 to Table 3-1, it can concluded that the horizontal rotation values due to wind loading for either the 1-year or 100-year storms are much smaller than the design criteria horizontal rotation values. This 1-year storm horizontal rotation is about 3 percent of the maximum design criteria horizontal rotation value, whereas the 100-year storm horizontal rotation is about 11 percent of the maximum value.

The service load combinations shown in Table 3-2 do not include any influence from the 100-year storm event. WSDOT must have designed for these effects in some other manner and further consultation with WSDOT is required concerning the design

Chapter 4

Basis of Joint Movements

The intent of this report is to develop new expansion joint details along the proposed light rail alignmentthe transition spans that accommodate the WSDOT design criteria movements as well as additional movements due to Light Rail Vehicle operations. The preliminary joint details provided in this report are based on fully satisfying the criteria described in Section 3.0 of this report. It is important, however, to first understand the basis of all the different movements that contribute to the WSDOT maximum design criteria movements in order to promote efficient and safe Light Rail Vehicle operations.

- The WSDOT design criteria joint movements include both normal and extreme events. WSDOT provided the tabulated Homer Hadley expansion joint movements for annual and ultimate events on July 26, 2007. The Homer Hadley expansion joint movement information provided by WSDOT is included in Appendix A. Based on this expansion joint data the design team concluded that:
- Maximum Longitudinal Translation Movement can be significantly bigger than the design criterion value of ± 24.5 inches under an ultimate event that includes Temperature, Longitudinal Braking Force from Live Load, and Wind and Wave effects. Therefore the Maximum Longitudinal Translation movement is limited by longitudinal restrainers at ± 18 inches. However, the design criterion Longitudinal Translation is specified as ± 24.5 inches.
- The Maximum Vertical Angle Change is shown as 0.8 degree for the annual event and 1.25 degrees for the ultimate event. However, the design criterion Vertical Angle Change is specified as 2.2 degrees.
- The Maximum Horizontal Angle Change is shown as 0.5 degree for the annual event and 1.0 degree for the ultimate event. However, the design criterion Horizontal Angle Change is specified as 1.1 degrees.

Since detailed calculations were not available from WSDOT, the correlation between the joint movements and the initiating events have not been fully determined. However, the design team has developed joint movements for many of the significant loadings under normal operating conditions. When the joint movements are correlated to different normal and extreme events, the Light Rail Vehicle Operating Plan can be formulated. Correlating normal and extreme events to the expected joint movement could result in a joint that can allow higher transit speeds over a vast majority of the system's life and still be operational at lower speeds during extreme events.

4.1 Light Rail Operating Plan

The Light Rail Operating Plan requires the evaluation of historical data (such as Lake Washington Hydrograph) for determining potential changes in lake level, weather data (including wind speeds for the 1-year and 100-years storms), and temperature variations for the temperature related joint movements.

As discussed in Chapter 3, historic and predicted temperature variations are required in order to evaluate the associated joint movements that contribute only to longitudinal translation movement. The longitudinal translation movement of the rails can be handled relatively easily by use of a combination of zero restraint fasteners and conventional split rail hardware. In addition, the zones of the longitudinal translation movement and expansion joint rotational movements will be separated from each other. Therefore, regardless of the historical temperature data (how frequent the maximum and minimum temperature variations occur) the WSDOT design criterion for longitudinal translation movement of ± 24.5 inches for the expansion joint can be achieved easily and will have no adverse effects on Light Rail operations.

The historic and predicted wind speeds are required to evaluate the 1-year storm and 100-year storm related joint movements that contribute mainly to the horizontal rotation movement of the expansion joints. We know that WSDOT's criteria state that the floating bridge will be closed if sustained 65 mph winds occur; we anticipate that the Light Rail operations would also be shut down. Based on our communication with Archie Allen of WSDOT, the Homer Hadley Floating Bridge has been closed to traffic only once during its service life. This closure happened during the 1993 Inauguration Day storm on January 20, 1993.

The change in lake level is the event that produces the largest vertical rotation and therefore has the greatest impact on the Light Rail operations. The WSDOT vertical rotation design criteria require a joint that is able to accommodate 2.2 degrees vertical rotation. This vertical rotation amount is equivalent to approximately 3.8 percent change in grade. That results in a significant change in grade. In ordinary circumstances, a vertical transition with this type of grade change would be accomplished over a length of several hundred feet. In the case of these joints, the transition length would occur in about 36 feet. With a 3.8 percent change in grade and 36 feet transition length the equivalent radius of the transition joint is about 938 feet. Based on Sound Transit Design Criteria Manual, a sag vertical curve having 938 feet radius can accommodate a Light Rail Vehicle speed of 20.5 mph. A crest vertical curve having same radius can only accommodate a Light Rail Vehicle speed of 15.3 mph. From the operating standpoint, these speeds are very low and would adversely effect run time of the Light Rail Vehicle.

Figure 4-1 shows the summary hydrographs for the Lake Washington from 1980 to 2007. Based on the hydrograph data, except for a period in the Fall of 1986, the lake level fluctuated between the Elevation 20.0 feet and 22.0 feet. The data shows that between September 27 and November 30, 1986, the lake level fell below the 20-foot elevation mark, reaching a low of approximately 19.4 feet for a continuous period of 14 days during that span of time. This is the only time that the Lake Washington level has fallen below Elevation 20.0 feet during the service life of the Homer Hadley Floating Bridge to date.

In the months of April, May, June and July, the lake level typically fluctuates between El. 21.0 feet and 22.0 feet, while during the months of December, January and February, it generally fluctuates between the El. 20.0 feet and 21.0 feet. During the remaining months of the year, lake level stays between the El 20.0 feet and 22.0 feet. Table 4.1 summarizes the historical Lake Level Fluctuations and corresponding vertical rotations at the expansion joints.

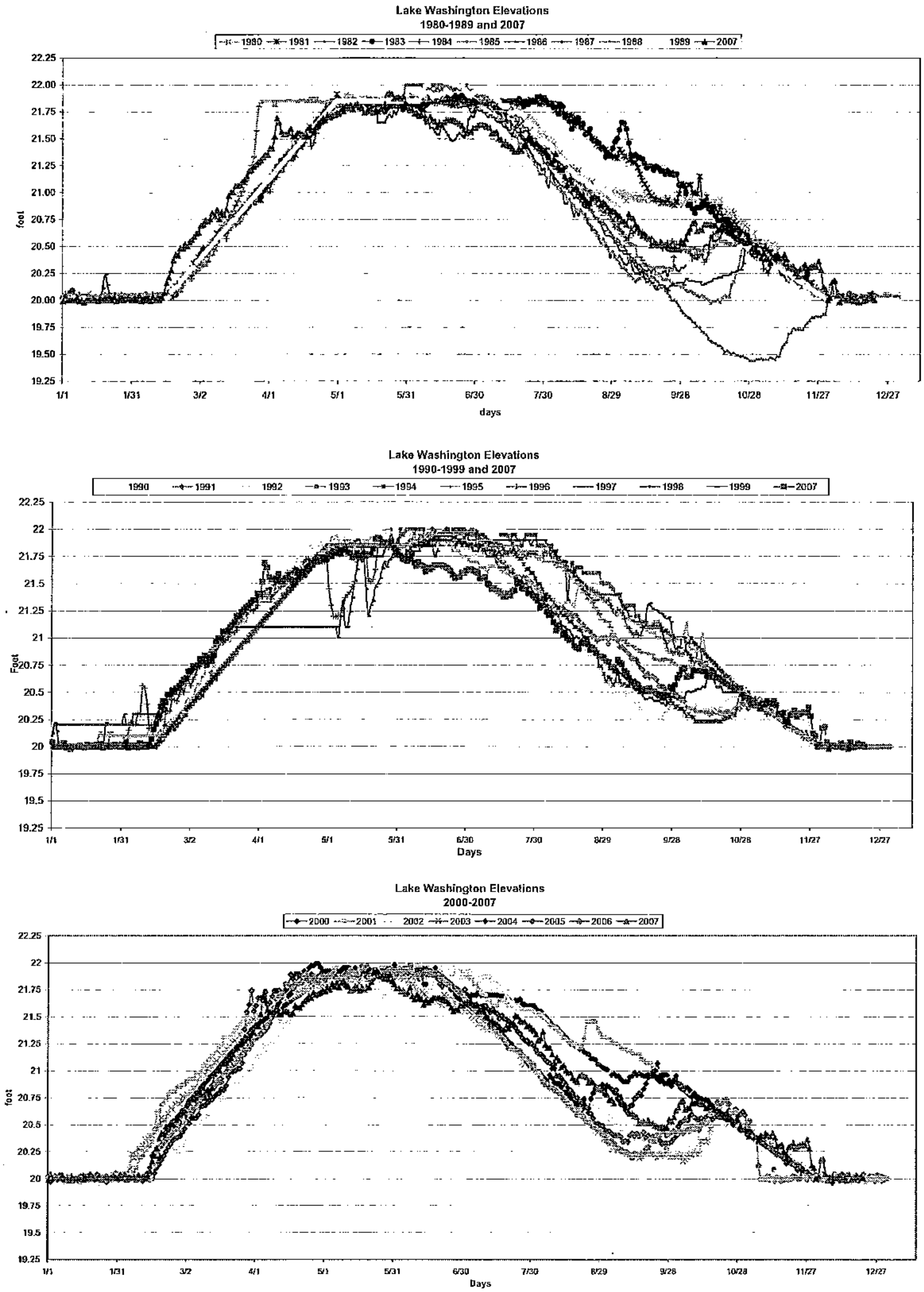


FIGURE 4-1
Summary Hydrographs for Lake Washington, 1980-2007

TABLE 4-1
Lake Washington Level Fluctuations

Month	Lake Level Fluctuation	Rise/Fall *(feet)*	VERTICAL ROTATION Rise θv *(deg)*	VERTICAL ROTATION Fall θv *(deg)*
January	20.0-21.0	0.0/-1.0	0.0	-0.306
February	20.0-21.0	0.0/-1.0	0.0	-0.306
March	20.3-21.8	0.8/-0.7	0.245	-0.214
April	21.0-22.0	1.0/0.0	0.306	0.000
May	21.0-22.0	1.0/0.0	0.306	0.000
June	21.0-22.0	1.0/0.0	0.306	0.000
July	21.0-22.0	1.0/0.0	0.306	0
August	20.3-21.8	0.8/-0.7	0.245	-0.214
September	19.8-21.8	0.8/-1.2	0.245	-0.368
October	19.4-21.6	0.6/-1.6	0.184	-0.490
November	19.4-20.8	0.6-1.6	0.184	-0.490
December	20.0-21.0	0.0/-1.0	0	-0.306

Other than the exception noted above for 1986, lake level fluctuates between El. 20.0 feet and 22.0 feet. Therefore, under normal operating conditions (Service Load Combination 1 – S1)and lake levels at El.20.0 feet of higher,, Light Rail Vehicle operating speeds listed in Table 4-2 were calculated using the Sound Transit Design Criteria Manual. The first row of numbers corresponds to the crest curve condition. The vertical angle value for the crest curve condition is maximum without the live load. The second row of numbers corresponds to a sag curve condition where the maximum vertical angle occurs with the live load.

TABLE 4-2
Operating Speed Limits for Service Group 1

Vertical Angle	Equivalent Grade	Equivalent Radius	Speed for Sag Curve	Speed for Crest Curve
(deg)	*Change (%)*	*(ft)*	*(mph)*	*(mph)*
0.306	0.5341	6741	-	41.1
0.543	0.9477	3799	41.3	-

Table 4-3 shows the calculated Light Rail Vehicle operating speeds when the 1-year storm movements are added to the above movements, under normal operating conditions (Service Load Combination 3 – S3) and lake levels at El.20.0 feet or higher.

TABLE 4-3
Operating Speed Limits Service Group 2 (Lake Washington level ≥ 20.0 feet El.)

Vertical Angle	Equivalent Grade	Equivalent Radius	Speed for Sag Curve	Speed for Crest Curve
(deg)	*Change (%)*	*(ft)*	*(mph)*	*(mph)*
0.324	0.5655	6366	-	39.9
0.561	0.9792	3677	40.7	-

Table 4-4 shows the calculated Light Rail Vehicle Operating speeds for periods when lake levels fall below El. 20 feet under Service Load Combination 3 (S3).

TABLE 4-4
Operating Speed Limits Service Group 3 (Lake Washington level < 20.0 feet El.)

Vertical Angle	Equivalent Grade	Equivalent Radius	Speed for Sag Curve	Speed for Crest Curve
(deg)	*Change (%)*	*(ft)*	*(mph)*	*(mph)*
0.508	0.8867	4060	-	31.9
0.745	1.3003	2768	35.3	-

It can be concluded that under normal operating conditions, including a 1-year storm event, 40 mph operating speeds can be achieved across the joints as long as the lake level fluctuates between elevations 20 and 22 feet. For occasional lake level drops below elevation 20 feet, the operating speed would need to be reduced to about 30 mph.

Again, the only time that the Lake Washington level has fallen below Elevation 20.0 feet during the service life of the Homer Hadley Floating Bridge was over a continuous 2.month period in the Fall of 1986.

Table 4-5 shows a tabulation of comparative LRT run times between the proposed Mercer Island and Rainier Stations across the floating bridge for operating speeds ranging from 25 to 55 mph. The system ridership analysis assumed a conservative operating speed of 25 MPH.

A discussion of the impacts of speed restrictions on LRT operations is provided in the Operations section of the East Link Conceptual Engineering Report.

TABLE 4-5
Mercer Island Floating Bridge Joints, Speed Increase Impacts
Running Time Rainier Flyover – Mercer Island Stations (A1 profile) different speeds over Floating Bridge Joints

Bridge Joint Operating Speed (mph)	Eastbound		Westbound	
	Time (m:s)	*Savings (m:s)*	*Time (m:s)*	*Savings (m:s)*
25	8:01	-	8:07	-
30	7:56	0:05	8:02	0:05
35	7:52	0:09	7:58	0:09
40	7:48	0:13	7:54	0:13
55	7:38	0:23	7:44	0:23

Chapter 5

Three-Beam Transition Details

5.1 Revision to the Transfer Beam Design

The previous conceptual design for the expansion joint at the ends of the transition spans has been modified in the following principal ways.

- the length has been increased from 24 feet to 36 feet, (at 24 feet, the Light Rail Vehicle was not able to satisfy the required minimum vertical clearance of 2 inches).
- by increasing the transition beam length, more rail clearance has been provided beneath the car, and
- additional supports have been provided at the end to minimize the distance between the rail fasteners when the structure is fully extended.

Numerous other changes have been made to simplify the design for both fabrication and maintenance. An isometric view of the revised transfer beam system is shown in Figure 5.1.

5.2 Transfer Beam Description

The purpose of the transfer beam is to support the rails under wheel load conditions from the trains while allowing the imposed deflections due to bridge movements to be distributed over a longer length of track. If the imposed deflections bend the rail over a short distance, the stresses in the rail will be very high. If these deflections are allowed to take place over a longer distance, the resulting stresses will be significantly reduced.

The use of a three-beam transfer system allows the imposed vertical angular deflections to be located at four locations. If the rail deflections were limited to just these four short locations, the rail stresses would still be too high. The system must allow enough vertical flexibility between the hinge locations so that the rail can curve smoothly over longer distances. The transfer beam system must also allow the horizontal deflections to take place over a reasonably long length so that the associated stresses in the rail are limited to an acceptable value.

At the same time, the design must provide a stiff enough system so that the deflections due to the passage of the loaded train are not so high that they adversely affect the performance of the train.

The revised design provides a workable solution that will be reviewed by trackwork engineers to confirm some of the design assumptions. Dimensions and other information in the following description are preliminary and subject to change as the design progresses. We will also coordinate with the rail fastener manufacturers as the design of these joints advances.

5.3 Controlling Dimensions

An isometric view of the revised transfer beam system is shown in Figure 5.1. As shown in Figure 5.2, Triple Beam Framing Plan, the total length of the system is 41'-8" and the length of the center beam is currently 22' – 6". The distance between the pivot points is 12 feet. The end beams extend an additional 8' – 5" past the end of the center beam. The top of the rail is approximately 1' - 3" above the deck elevation.

A method to accommodate the transfer beams is to provide a recess in the deck. The current layout requires a recess approximately 9 inches deep as shown in Figure 5.3. The recess is required to maintain the standard plinth dimensions and to keep the added dead load as low as possible. However, providing a recess may require extensive modifications and may become costly. The other alternative to the recess is to gradually increase the plinth heights in the proximity of the expansion joints. Both options will be investigated in future studies.

5.4 Structural Description

The transfer beams are currently fabricated structural steel plate girder sections approximately 20 inches deep (Figure 5.3). The flanges are 6 inches wide and 1 inch thick. The center-to-center distance between the center and end beams is 8 inches.

The ends of the beams, which are supported by the deck, have curved bearing plates to accommodate the vertical rotation of beam ends relative to the support structure. The bearing plates will bear on sliding bearings, which accommodate the longitudinal movement of the transfer beams. Numerous options are available for these bearing surfaces and will be finalized at a later date.

Transverse guides are attached to the supporting structure as shown in Figure 5.4. These steel fabrications provide a bearing surface to resist transverse loads. They also keep the beams from rotating. The interface between the center and end beams is also equipped with vertical bearing plates so that the transfer beam system maintains a constant width. The faces between the beams will be provided with a bearing material to allow for some deflection in the joints and a bearing surface to reduce the friction.

The transverse guides have to be long enough to allow the end of the beams to move through their entire range.

These transverse guides must have the capacity to resist both lateral loads from the train as well as loading due to the transverse rotation of the transition span relative to the adjacent structure. The pair of transverse guide sets at each end of the transfer beam apply a moment to the center area of the structure, which then will bend in a curve. This curve must be able to distribute the transverse bending in the rails over sufficient length to minimize the stresses in the rails. Some flexibility may be required in the bearings at the transverse guides to control the horizontal bending.

The pivot between the end and center beams is provided by a transverse shaft and bearings located in each beam. A bearing surface is located between the beams at this location to resist transverse loads. The bearing assembly at the two beams must also be able to resist tension so that the beams remain in contract.

Transverse beams between the beams on each side are formed by plate girder sections. They support the rail fasteners between the side beams. The plate section provides both the necessary structural capacity and fixity against rotation of the side beams.

Due to the longer length of the transfer beams, there are now more than two rail fasteners at each beam section. Some vertical flexibility must be provided so that the rails can maintain a relatively smooth curve due to angular deflections of the joint. Some of this vertical flexibility can be provided in the rail fasteners. If more is needed, springs can be installed between the top of the crossbeam.

Due to the longer length of the transfer beams, their deflection under live loads will become a bigger issue. At this time, an effort has been made to minimize the depth of the beams. As the design of the ends of the transfer span is done, it may be possible to increase the depth of the beams without unacceptable consequences. It may also be possible to accommodate some of the live load deflections by cambering the transfer beams. The most practical way may be to adjust the rail geometry be shimming between the rail fasteners and the top of the crossbeams.

The typical spacing of cross beams and rail fasteners is 29 inches. The center-to-center distance between the last rail fastener located on a cross beam and the first rail fastener located on a concrete plinth is currently 24 inches. This allows the transfer beam to move 13 inches toward the support structure without interference between the fasteners. When the relative movement between the transfer beam and the structure expands, the center-to-center distance between fasteners is 37 inches. This spacing is somewhat larger than normal usage, but this condition will rarely occur, if ever, since it represents the maximum design event movement of the joint and not necessarily the actual movement. The frequency of these movements is the information that will be found by instrumenting the bridge joints at the transition spans. During final design, the frequency of this movement of the joint can be factored into the analysis of the joint and rails.

The best condition would be if the transfer beams remained centered between the adjoining structures. If the transfer beams are allowed to move toward one extreme or the other, it results in the spacing between rail fasteners being larger than necessary. The transfer beam structure will be provided with stops so that it is not necessary to keep it centered in the gap.

Figure 5.5 shows a preliminary concept for the centering device for the three beam support. Due to the multiple motions required for this structure, a cable system is being considered. The cables would be anchored on one structure, pass 180 degrees around a sheave on the support beams, and return to the end of the opposite structure. When one structure moves longitudinally with respect to the other structure, the center support beams are constrained to move one-half the distance and remain centered. Some flexibility would be build into the cables so that they can accommodate the other motions required of the joint. The flexibility can be provided either by building some slack into the cables, or by providing springs at the anchorages.

There are other centering systems such as linkages that should also be considered for the final design.

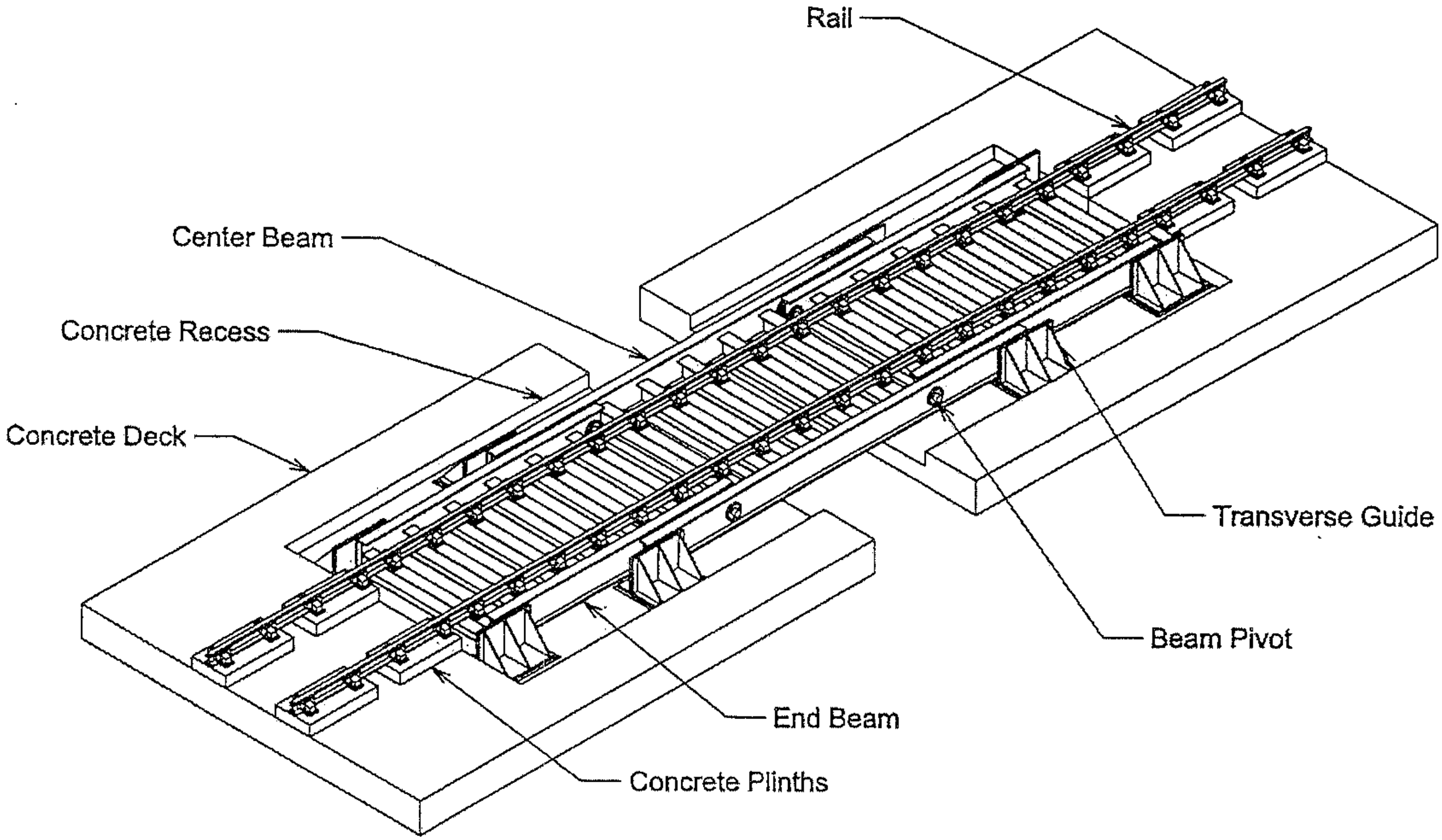


FIGURE 5-1
Isometric View of Transfer Beam System

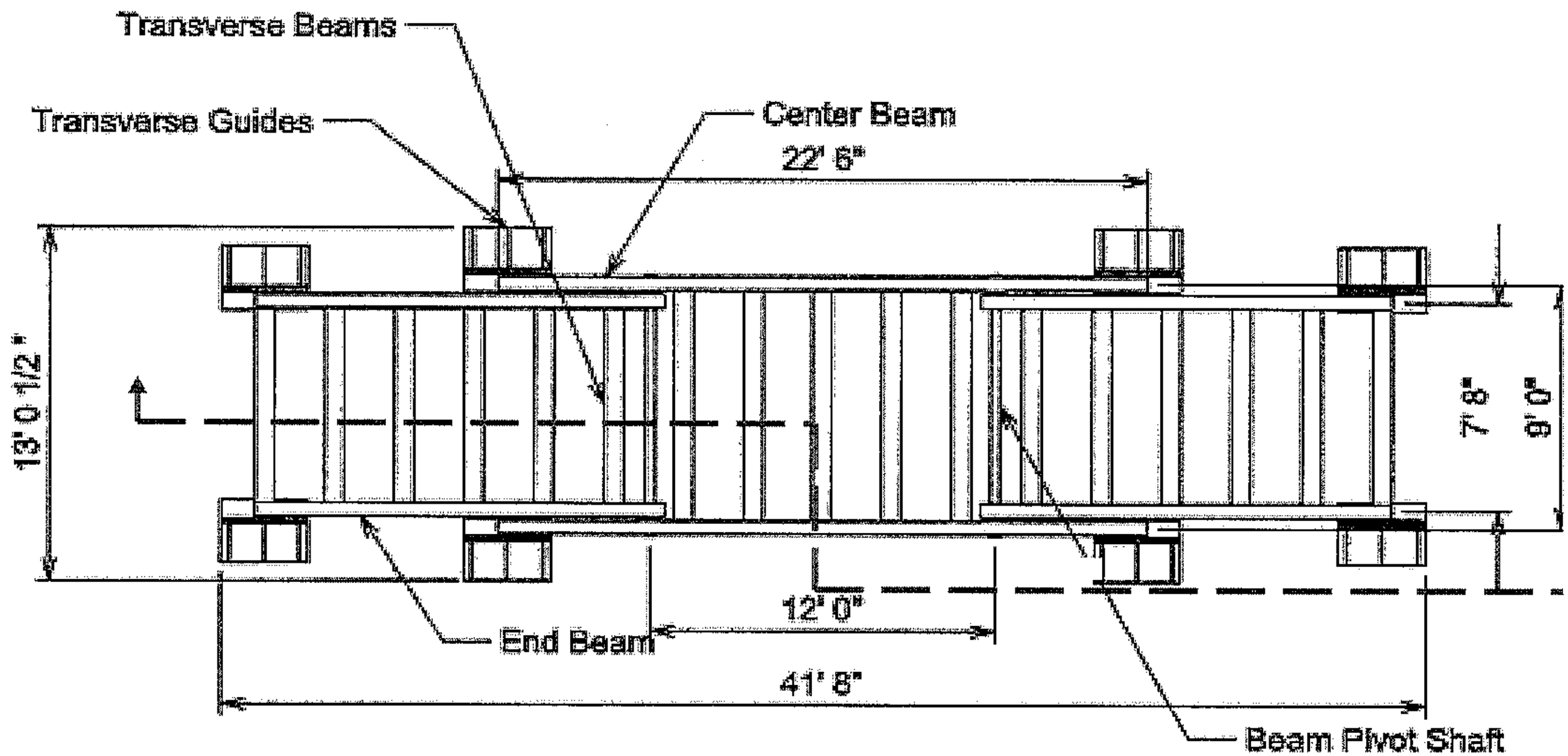


FIGURE 5-2
Triple Beam Framing Plan

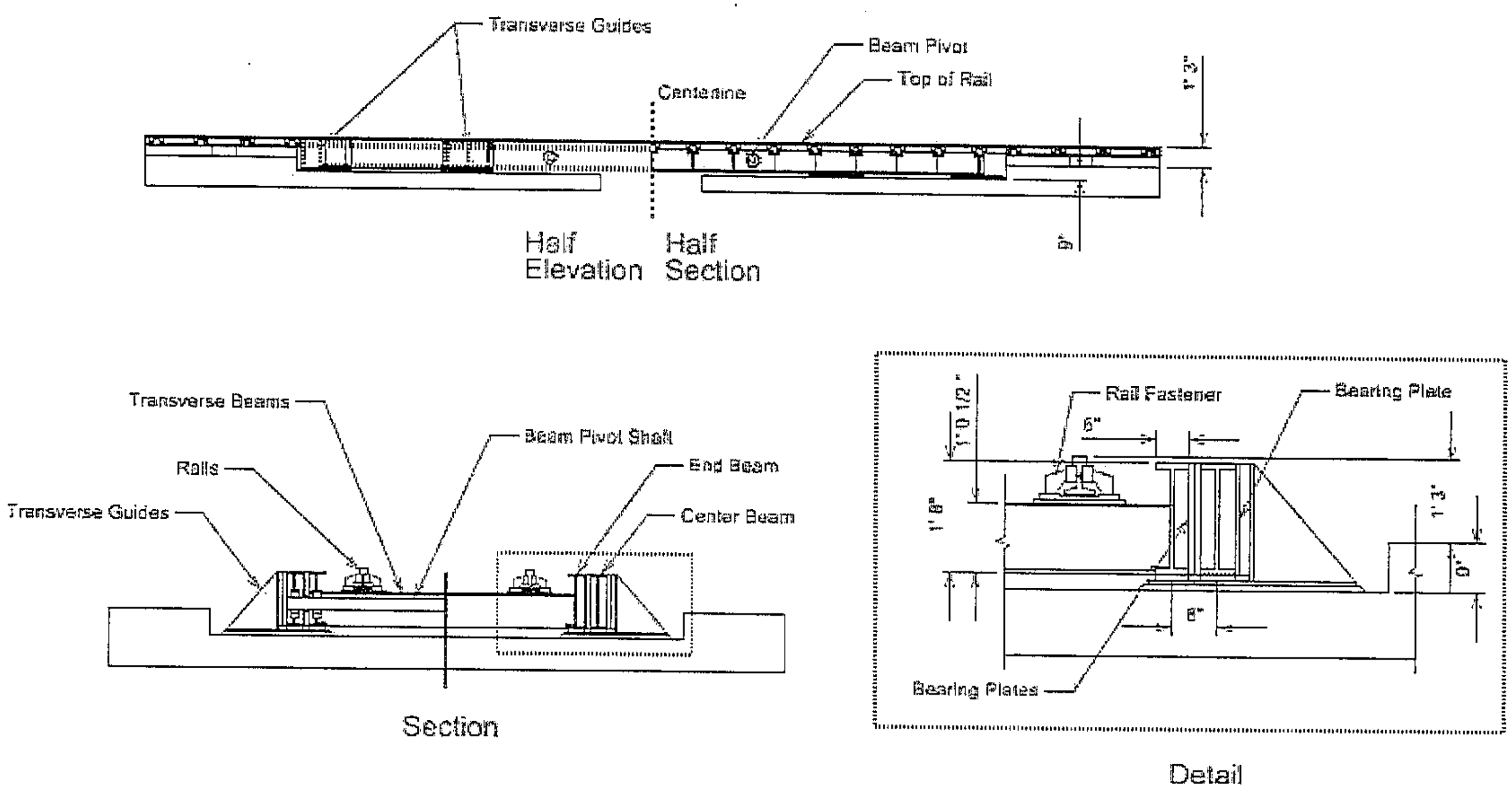


FIGURE 5-3
Triple Beam System Sections

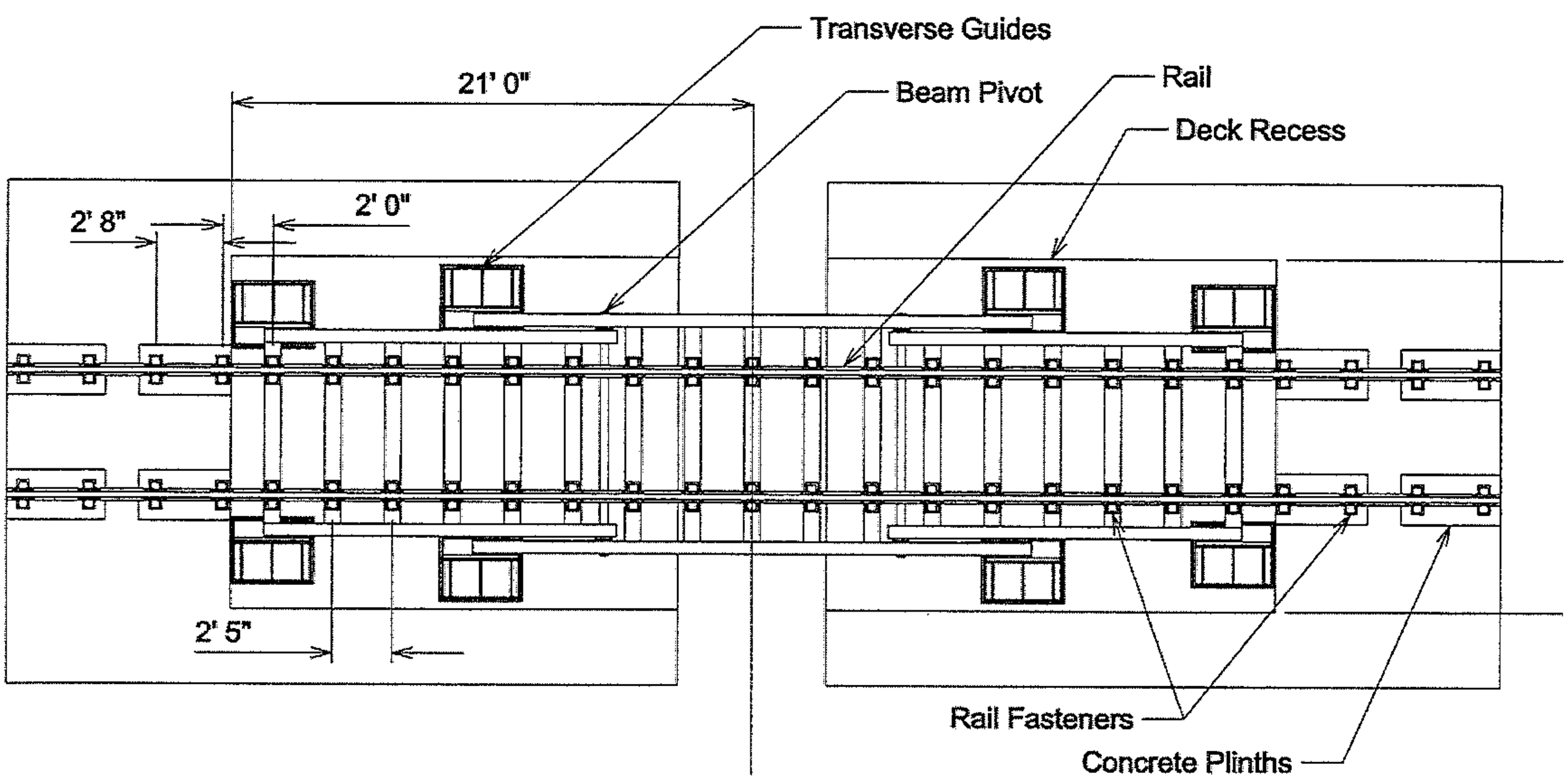


FIGURE 5-4
Triple Beam Overall Plan

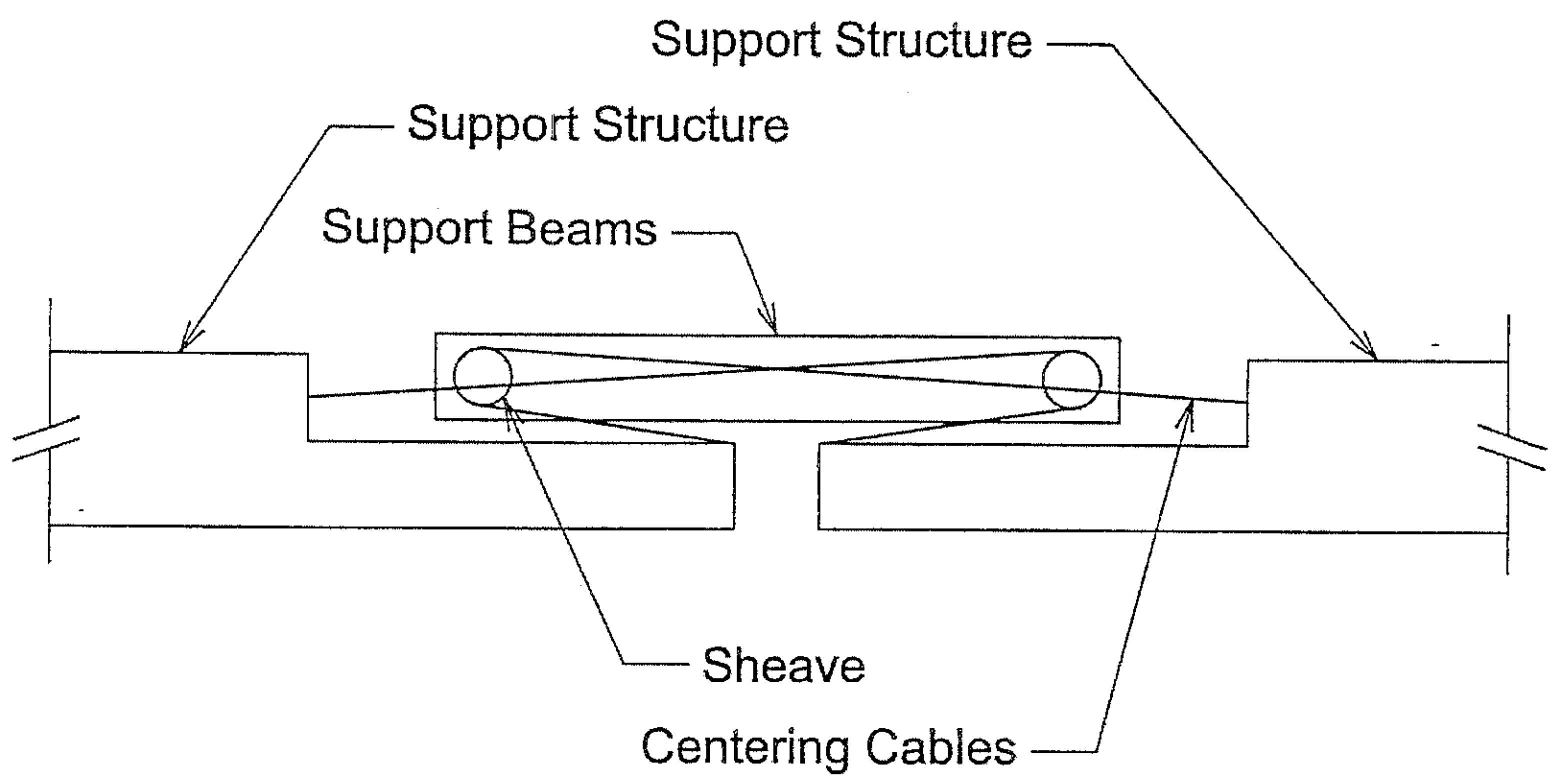


FIGURE 5-5
Centering Device

Chapter 6

Conclusions

A preliminary structural analysis was performed on the three-beam transition concept to develop the structural details to the 10 percent design level. The previous conceptual design has been modified to simplify the three-beam transition concept for both fabrication and future maintenance. Preliminary member sizes were determined for the supporting beams, support mechanism between the transition beams and the running rails, and support mechanism between the transition beam and the bridge decks.

It has been further verified that the simplified three-beam transition concept can accommodate all necessary bridge and LRT movements and will provide for the uninterrupted Light Rail Transit service across the I-90 (Homer Hadley) Floating Bridge.

A detailed evaluation of the contributions of different load conditions on the maximum design joint movement criteria set by WSDOT was undertaken, along with a preliminary assessment of the Light Rail Operating speeds under normal operating conditions (including a 1-year storm event). It is concluded that a Light Rail Vehicle can be safely operated at speeds up to 40 mph as long as the change in the lake level stays between the elevations 20.0 and 22.0 feet. During the occasional lake level drops below the elevation 20.0 feet, the operating speed may need to be reduced to 30 mph for safe Light Rail Vehicle operation across the transition spans. As the East Link Project advances into the preliminary and final design phases, the consultant team will re-evaluate the proposed operating speeds in more detail. If required, the models will be further refined to investigate the operational aspects of the expansion joints in order to insure the structural integrity and comfort of the riders.

APPENDIX A

WSDOT Provided Joint Movement Information

Following Homer Hadley Floating Bridge transition spans expansion joint movement tables and sketches were provided by Dylan Counts of WSDOT on behalf of Patrick Clarke in July 26, 2007.

From: Counts, Dylan [mailto:CountsD@wsdot.wa.gov]
Sent: Thursday, July 26, 2007 8:59 AM
To: Billen, Don; Comis, Sue; Koester, Roger
Cc: Greco, Theresa; Becher, David
Subject: FW: Homer Hadley Joint Movements

Floating Bridge information from Patrick Clarke as discussed in our Friday conference call.

Dylan

From: Clarke, Patrick
Sent: Wednesday, July 25, 2007 4:40 PM
To: Gren, Theresa
Cc: Counts, Dylan; Messmer, Tony
Subject: FW: Homer Hadley Joint Movements

Here is the table of the calculated Homer Hadley expansion joint movements and the installation report for the expansion joint on the Lacey V. Murrow Floating Bridge, the installation and the joint were very similar to the Homer Hadley Floating Bridge. As we discussed these are for information only for Sound Transits engineers looking at the track system and they agreed that they would have a wind and wave analysis done that focuses specifically on the LRT operational issues as it relates to the allowable stresses on the bridge and the overall operational safety of the bridge. In our conference call last Friday it sounded like they are planning on pursuing a plan to instrument the joints on the bridge to get more accurate data on the movements of the joint due to seasonal lake level and temperature fluctuations to compliment the wind and wave analysis.

Let me know if there is anything else that you need.

Patrick

Motion of Floating Structure	Max. Longit. Design Movement of Exp. Jt.	Maximum Angle Change	Loads Contributing to Movement								
			Temp.	Damage (Flooding)	Live Load	Wind & Wave Longit.	Wind & Wave Transv.	Lake Level	Cable Tension	Extra Dead Load	Earthquake
LONGIT. HORIZ.	±1.5' (1)	0° Horiz. 0.8° Vert. (9)	+0.9(open) -0.6(close) (5)		±1.50'	±2.03'					±0.7' (3)
TRANSV. HORIZ.	±0.35' (2)	7° Horiz. (6) 0° Vert.					±0.35' (2)				Small
VERT. {RISE / FALL	+0.04(open) -0.20(close)	0° Horiz. 1.25° Vert. (7)			-0.02'			+0.04' -0.18'			
ROTATION (ROLL) (4)	± 0.14	0.5° Horiz. (8) 0.5° Vert. (8)			±0.14		±0.02'				

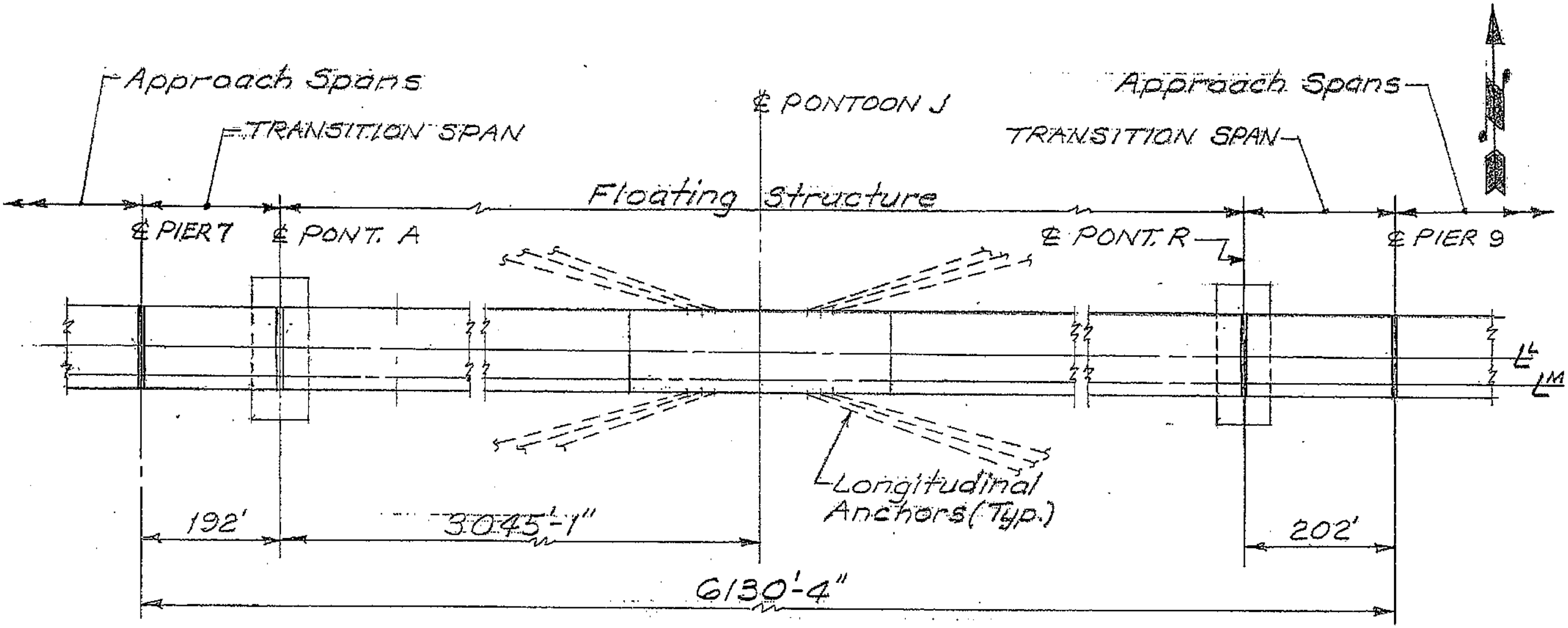
(1) Limited by Longitudinal Restrainers.
(2) Longit. movement at curb lines on LM Structure. (20' from ℄ LM)
(3) EQ movement due to response of approach spans.
(4) Roll produces vertical and transverse motions and are a maximum at locations on the roadway which are farthest from C.G. of pontoon.
(5) Effects of approach spans included.
(6) See Fig. 4
(7) See Fig. 5
(8) See Fig. 6
(9) See Fig. 8

JOINT MOVEMENT
PONTOON A AND R
ULTIMATE EVENT

Motion of Floating Structure	Max. Longit. Design Movement of Exp. Jt.	Maximum Angle Change	Loads Contributing to Movement								
			Temp.	Damage (Flooding)	Live Load	Wind & Wave Longit.	Wind & Wave Transv.	Lake Level	Cable Tension	Extra Dead Load	Earthquake
LONGIT. HORIZ.	$\pm$ 1.5' △1	0° Horiz. 0.8° Vert. △9	+0.9' (open) -0.6' (close) △5		$\pm$ 0.77'	$\pm$ 0.40'					$\pm$ 0.7' △3
TRANSV. HORIZ.	$\pm$ 0.06' △2	0.2° Horiz. △6 0° Vert.					$\pm$ 0.06' △2				Small
VERT. {RISE / FALL}	+ 0.04' (open) - 0.05' (close)	0° Horiz. 0.4° Vert. △7			- 0.02'			+0.04' -0.05'			
ROTATION (ROLL) △4	$\pm$ 0.14'	0.5° Horiz. △8 0.5° Vert. △8			$\pm$ 0.14'		Small				

△1 Limited by Longitudinal Restrainers.
△2 Longit. movement at curb lines on LM Structure. (20' from ℄ LM)
△3 EQ movement due to response of approach spans.
△4 Roll produces vertical and transverse motions and are a maximum at locations on the roadway which are farthest from C.G. of pontoon.
△5 Effects of approach spans included.
△6 See Fig. 4
△7 See Fig. 5
△8 See Fig. 6
△9 See Fig. 8

JOINT MOVEMENT
PONTOON A AND R
ANNUAL EVENT



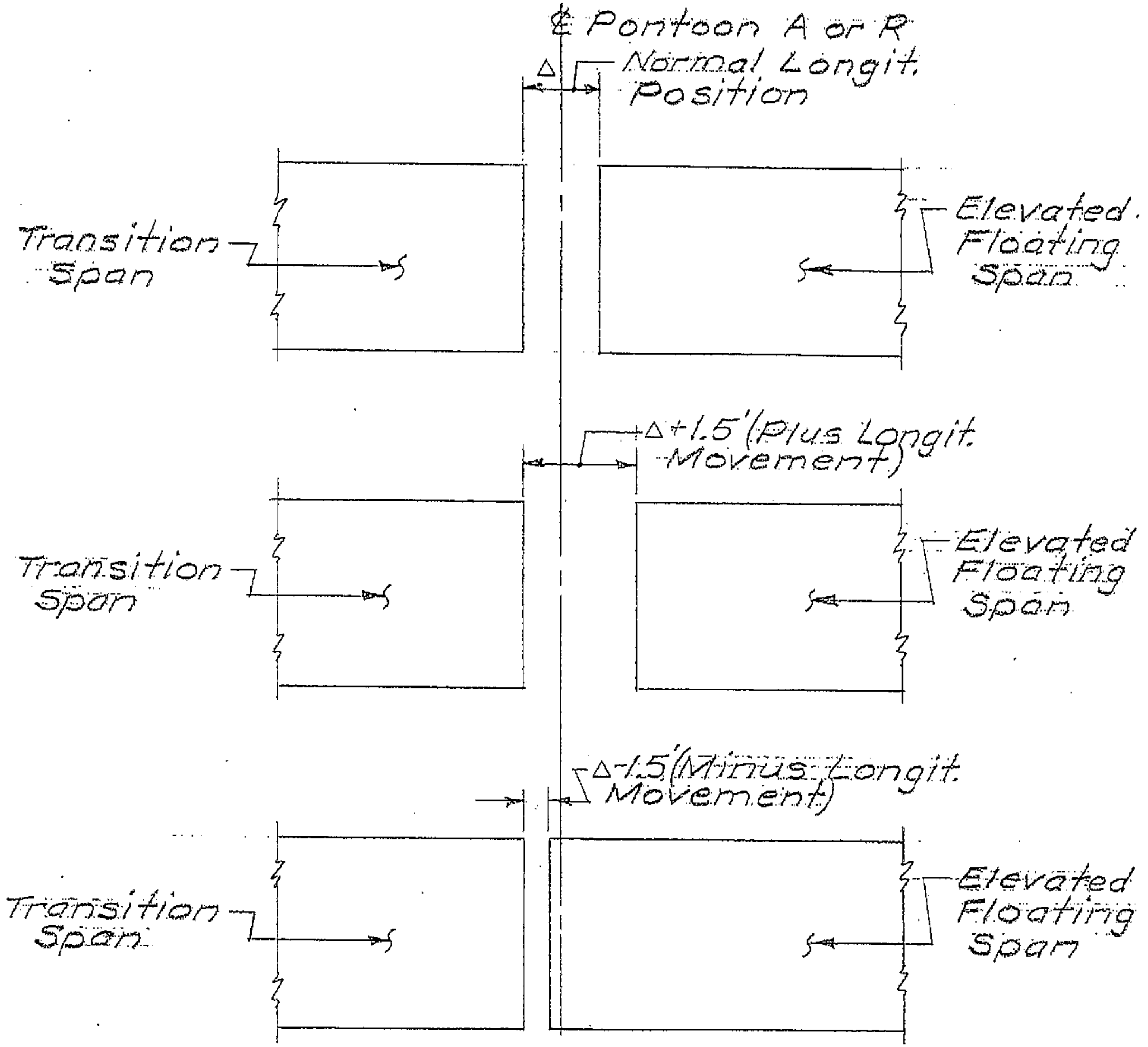
PLAN

3RD Lake Wash. Floating Bridge

Expansion Joint - Pontoon A & R - Design Motions

Longitudinal Motion → ± 1.5'
Transverse Motion → ± 3.0'
Vertical Motion → 0.8' Rise
3.8 Drop

} Motion of Floating Structure with respect to Approach Span.



PLAN - LONGITUDINAL MOTION OF ±1.5'

APPENDIX B

Homer Hadley Floating Bridge Expansion Joint Quantities

QUANTITIES		
	Quantity	Unit
Remove Part of Existing Modular Expansion Joint	56	LF
Remove Existing Deck Concrete	62	CY
Structural Steel for Structural Modifications	22500	LB
Superstructure Concrete (5000psi) for Structural Modifications	68	CY
Structural Steel for Expansion Joint	85000	LB

Eastside HCT Corridor
Existing I-90 Bridges
Seismic Vulnerability Study
Final Conceptual Report

Prepared for:
Sound Transit

Prepared by:
Sound Transit East Link Project Team

January 2008

Quality Tracking: (Keep on front page)

Resource Lead: Ahmet Ozkan, P.E. **Date Submitted for review:** (Date)

Senior Reviewer: Dan Russell, P.E. **Review completion date:** (Date)

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Editor: (Name/wet initials) **Date finalized for client review:** (Date)

EAST LINK PROJECT

Existing I-90 Bridges Seismic Vulnerability Study Final Conceptual Report

January 2008

SOUNDTRANSIT

CENTRAL PUGET SOUND REGIONAL TRANSIT AUTHORITY

SOUND TRANSIT

SOUND TRANSIT EAST LINK PROJECT
Phase 2

Existing I-90 Bridges Seismic Vulnerability Study Final Conceptual Report

Prepared for:
CH2M HILL, Inc. and Sound Transit

Prepared by:
INCA Engineers, Inc.

January 2008

Translation services and information in accessible formats are available upon request by calling 1.800.201.4900 (voice) or 206.398.5410 (TTY).

For more information about the SR90 Floating Bridge (Homer Hadley) Seismic Vulnerability Study Final Conceptual Report call (refer to specific community relations coordinator as appropriate) or write Sound Transit, 401 South Jackson Street, Seattle, WA 98104-2826. You may also e-mail Sound Transit at main@soundtransit.org, visit our Web site at www.soundtransit.org or call our toll free information line at 1-800-201-4900.

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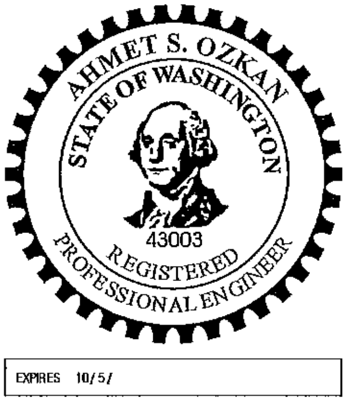
Exhibits

CERTIFICATE OF ENGINEER

The work contained herein was prepared under the supervision and direction of the undersigned.



Ahmet Ozkan, P.E.
Chief Bridge Engineer

Chapter 1

Executive Summary

This report describes results of seismic vulnerability analysis performed on the existing I-90 bridges (excluding floating spans) between downtown Seattle and Bellevue to evaluate the impacts of the Light Rail Vehicle loading and related structural modifications, and current and future seismic design specification requirements on the seismic performance.

1.1 Impacts of the Light Rail Vehicle Loading

As-built plans of the existing bridges include provisions for a future Rapid Transit Loading consisting of a maximum of eight 70-foot-long vehicles with a load of 100 kip each, which is equivalent to a uniform load of 1.43 kip/foot. The current Sound Transit Light Rail Transit loading consists of a maximum of four 91.9-foot-long vehicles with a load of 147.6 kip each, which is equivalent to a uniform load of 1.61 kip/foot. The difference between the as-built plans loading to current Sound Transit loading amounts to an about 12.5 percent increase in the live load. For the existing bridge superstructures, the local and global effects of the live load increase were studied by KPFF for WSDOT. In multiple reports, KPFF included mitigation measures for the superstructure elements that required strengthening under AASHTO Group I Load Combinations.

Presently Sound Transit Design Criteria Manual addresses seismic design for new structures but does not define an approach for existing structures. In addition to that, Sound Transit aerial structures design practice differs from WSDOT bridge design practice, in particular, in seismic load combination. Sound Transit includes Light Rail Vehicle mass in their seismic analysis and design of new aerial structures, whereas, WSDOT ignores the live load mass in the seismic load combinations for both new and existing bridges. When included, live load inertia effects increase the seismic loads significantly.

In their May 10, 2007 letter (Appendix A)., WSDOT established a new Seismic Retrofitting Criteria for the existing WSDOT bridges. The existing structures seismic evaluation are based on WSDOT letter proposed retrofit criteria including WSDOT's current practice of using 50% of the vehicular live load in Seismic Load Combinations. In addition, 100% of the Light Rail Vehicle AW4 loading, without impact, acting on one track only is included in Seismic Load Combinations. Both vehicular and Light Rail live loads are included as gravity loads. Per AASHTO LRFD Extreme Event-I load combination and WSDOT's current practice, live load mass is ignored in the seismic analysis.

Since Light Rail Vehicle mass is not included in the seismic analysis, and the live load is accounted as gravity load (no inertia effects), the 12.5 percent increase in live loading has negligible effect on seismic load combinations of the substructure members.

1.2 Impacts of the Light Rail Transit Related Dead Loads

As-built plans of the existing bridges include provisions for a future Rapid Transit Loading related dead loads. In the as-built plans, the dead load allowance is 895 pounds/foot/track or 1,790 pounds/route-foot. The new dead loading conditions created by Light Rail Transit conversion of the existing bridges include updated trackwork cofiguarations, and structural modifications.

The additional dead loads vary among the existing bridges; 920 pounds/route-foot (for D2 Roadway exclusive LRT, Transition Spans and Approach Spans), 1,470 pounds/route-foot (for Rainier Avenue Overcrossing and East Channel Bridge) and 2,460 pounds/route-foot (for D2 Roadway Bus and LRT joint operation alternative). Therefore, the LRT modified structures will have a total mass that is a couple of percent (1 to 7 percent) less than the total mass provisions of the as-built bridges original design except for the D2 Roadway bus and LRT joint operation alternative. In this case, the LRT modified structure will have a mass that will be a couple of percent (2 to 4 percent) higher than the total mass assumed in the original design. However, the increase in the mass (2 to 4 percent) for the D2 Roadway bus and LRT joint operation alternative is small enough to being considered as "negligible by inspection".

Since the Light Rail Transit related dead loads are within a couple of percentage point of the as-built plan dead load provisions for the existing bridges, there are no seismic demand increase due to structural modifications that are required for Light Rail Transit related structural modifications.

1.3 Impacts of the Changes in the Seismic Design Criteria

The existing I-90 bridges included in this study have been designed during the 70s and mid 80s. Design and construction practices of the day (especially 1970's and early 1980's) did not include special seismic design provisions comparable to the current seismic design requirements. Therefore, the real impact on the seismic demand of the existing bridges is due to the changes in the seismic design criteria and detailing practices not the live load increase due to Light Rail Vehicle or LRT required structural modifications.

All the existing bridges were designed by using *ATC-6, Seismic Design Guidelines for Highway Bridges, Applied Technology Council, October 1981,* with a maximum expected peak ground acceleration (PGA) of 0.20g, except, the East Channel Bridge which is designed for a PGA of 0.5g and corresponding response coefficient of 20 percent of the total mass.

The current WSDOT seismic design criteria for both <u>new bridges</u> and <u>retrofits</u> are based on 2002 USGS Maps (10% probability of exceedance (PE) in 50 years, return period of 475 years). Whereas, the current Sound Transit seismic design criteria for <u>new aerial structures</u> is based on two level performance criteria and requires a significantly higher level of performance (Operational Earthquake with 150 years return period and Maximum Earthquake with 2500 years return period).

As owner of the existing bridges, WSDOT has indicated that they anticipate adopting a new seismic design criteria based on a 1,000-year return period as detailed in *AASHTO Guide Specifications for LRFD Seismic Bridge Design, Subcommittee for Seismic Effects on Bridges, T-3, March 2007* and FHWA Manual titled *Seismic Retrofitting Manual for Highway Structures, December 2006* instead of the current design criteria using the 475-year return period earthquake.

A comparison of the original, 475-year and 1,000-year event response coefficients are as shown in the following Figure.

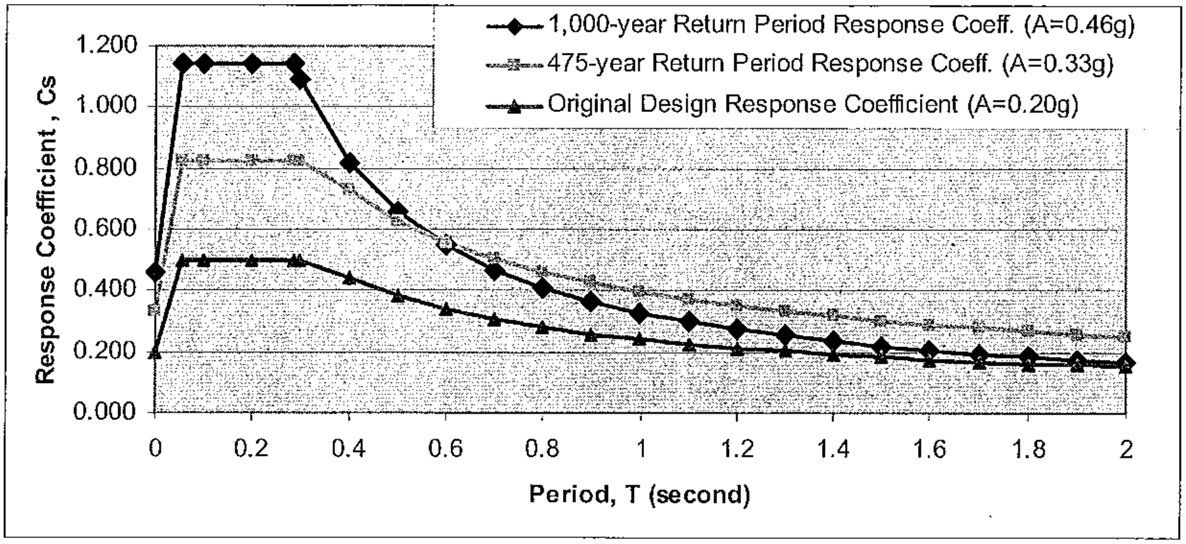


EXHIBIT 1-1
Comparison of 475-year and 1,000-year Event Coefficients

The PGA value selected as 0.33g for 475-year event based on the location of the existing bridges. The PGA value calculated as 0.46g for 1,000-year event based on new criteria equations. Based on these numbers, the ratio of 475-year event to original event is about 1.65. This means 65 percent increase in the seismic demand from the original design criteria event and it is constant for all structural periods. On the other hand, the ratio of 1,000-year event to original event is about 2.30 (130 percent increase) times for short period structures and it decreases as the period of the structure increases (the ratio is about 1.20 (20 percent increase) for a period of 1.5 seconds).

1.4 Conclusions/Recommendations

Based on design team's seismic evaluation study on the existing bridges, this report concludes that:

- The effect of the Live Load increase on substructure elements due to Light Rail Vehicle is negligible and does not create a seismic load demand increase from the original design.
- Since the Light Rail Transit related dead loads are within plus/minus couple of percentage points of the as-built plan dead load provisions for the existing bridges, there are no seismic demand increase due to Light Rail Transit related structural modifications.
- Since there are significant changes in the seismic design criteria and detailing practices during the last 30 years, Sound Transit and WSDOT need to determine whether to retrofit the existing I-90 bridges that will be utilized by the Light Rail Vehicles to the current criteria. If the new criteria is utilized, there is significant seismic demand increase on the existing bridges. However, the new changes in the Seismic Bridge Design and Retrofitting provisions directs the designers explicitly for a larger seismic event (1,000-year), but refine the provisions to reduce the conservatism. When the new seismic design and retrofitting provisions are published and included as part of the AASHTO LRFD and WSDOT Bridge Design Manual, the existing bridges should be re-evaluated based on refined analysis methods to determine if the seismic retrofit is required.

Chapter 2

Introduction

Use of the I-90 HOV corridor for the East Link Light Rail Transit connection between Seattle and Bellevue requires the use of existing WSDOT bridges. These existing bridges were designed during the 70s and mid 80s. Design and construction practices of the day (especially 1970's and early 1980's) did not include special seismic design provisions comparable to the current AASHTO seismic design requirements. Therefore, these existing bridges require an evaluation for the current seismic loads.

This report describes results of seismic vulnerability analyses performed on the existing I-90 bridges between downtown Seattle and Bellevue to identify critical structural components that could be damaged by a design level earthquake.

This report also analyses these existing bridges without Light Rail Vehicle operations and with Light Rail Vehicle operations based on WSDOT recommended seismic design and retrofit criteria to identify the differences under different live loading conditions.

This study includes the following structures:

- Transition Spans (Homer Hadley?)
- Approach Structures (Homer Hadley)
- D2 Roadway Viaduct
- Rainier Avenue Overcrossing (Bridge #222)
- East Channel Bridge

Floating Bridge is excluded from this study since the original WSDOT design criteria of Homer Hadley Bridge states that *"The pontoon need not be designed for earthquake generated forces. It is felt the water will provide sufficient dynamic absorption capacity to minimize any earthquake loads"*.

2.1 References

The following publications are referenced in this report.

1. I.G. Buckle, et .al., Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges by MCEER and FHWA, 2006

2. M.J.N. Priestley, F. Seible and G.M. Calvi, Seismic Design and Retrofit of Bridges, John Wiley and Sons, Inc., 1996

3. J. Kapur, Washington State's Bridge Seismic Retrofit Program

Chapter 3

Seismic Criteria

Use of the I-90 HOV corridor for the East Link LRT connection between Seattle and Bellevue requires the use of existing WSDOT bridges. These existing bridges may need to be strengthened for current AASHTO seismic loads using the 475-year recurrence level earthquake. Presently Sound Transit Design Criteria addresses seismic design for new structures but does not define an approach for existing structures. WSDOT has indicated in their letter dated May 10, 2007 that they anticipate adopting a seismic design criteria based on a 1000-year recurrence as detailed in FHWA Manual Seismic Retrofitting Manual for Highway Structures instead of the current design criteria using the 475-year recurrence level earthquake.

3.1 Seismic Retrofit Level Earthquake

FHWA's new 2006 Seismic Retrofitting Manual for Highway Structures is a major revision of FHWA's 1995 Seismic Retrofitting Manual for Highway Bridges. New information added includes current advances in earthquake engineering, field experiences with retrofitting highway bridges, and the performance of bridges in recent earthquakes in California and elsewhere.

The new manual introduces a performance-based retrofit philosophy that is similar to the one used for the performance-based design of new buildings and bridges. Bridge performance criteria are given for two earthquake ground motions with return periods of 100 years and 1,000 years, respectively. The Lower Level earthquake having a 50% probability of exceedance in 75-years (corresponds to a return period of about 100 years) and the Upper Level earthquake having a 7% probability of exceedance in 75-years (corresponds to a return period of about 1,000 years). A higher level of bridge performance is required for the event with the shorter 100-year return period than for the one with the 1,000-year return period. Retrofit criteria are recommended according to bridge performance and anticipated service life. A more rigorous performance is required for important or new bridges, and a lesser level performance for standard bridges nearing the end of their useful life.

In this study, the seismic load analysis is performed by using current AASHTO LRFD and Current, WSDOT Bridge Design Manual provisions considering 475-year recurrence level earthquake. However, WSDOT has indicated in their letter dated May 10, 2007 that they anticipate adopting a seismic design criteria based on a 1,000-year recurrence event, and it will be effective by January 2008. Based on WSDOT's May 10, 2007 letter on Seismic Design Issues and Sound Transit's directions during August 15, 2007 Seismic Design Criteria Meeting, in this study, we included 1,000-year recurrence level event in addition to current seismic retrofit level earthquake of 475-year recurrence event for evaluating the seismic vulnerabilities of the existing bridges.

TABLE 3-1
Seismic Design Criteria Comparison

SEISMIC DESIGN CRITERIA COMPARISON	
Existing ST Criteria for New Aerial Structures	ST DCM assumes the design life of the bridges as 100-year. ST DCM specifies a two-level earthquake design: • Operational with 50% probability of exceedance in 100 years (150-year return period) • Maximum with 4% probability of exceedance in 100 years (2,500-year return period)
Existing ST Criteria for Existing Aerial Structure Retrofits	Currently, ST does not have any provisions for seismic retrofit.
Existing WSDOT Criteria for New Bridges	WSDOT BDM-LRFD specifies a single level earthquake: • 10% probability of exceedance in 50 years (475-year return period)
Existing WSDOT Criteria for Existing Bridge Retrofits	WSDOT BDM-LRFD specifies a single level earthquake: • 10% probability of exceedance in 50 years (475-year return period)
Existing AASHTO LRFD Criteria for New Bridges	AASHTO-LRFD specifies a single level earthquake: • 15% probability of exceedance in 75 years (475-year return period) AASHTO LRFD has provisions for some bridges that must remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after a large earthquake, e.g., a 2500-year return period event (3% probability of exceedance in 75 years). These bridges regarded as critical structures.
Existing AASHTO LRFD Criteria for Existing Bridge Retrofits	Currently, AASHTO-LRFD does not have any provisions for seismic retrofit.
Existing FHWA Criteria for Existing Bridge Retrofits	FHWA specifies a two-level earthquake design: • Lower Level earthquake with Operational with 50% probability of exceedance in 75 years (100-year return period) • Upper Level earthquake with 7% probability of exceedance in 75 years (1,000-year return period)
Proposed AASHTO LRFD Criteria for New Bridges (WSDOT BDM-LRFD will adopt it in 2008 for both new bridge designs and existing bridge retrofits)	AASHTO-LRFD specifies a two-level earthquake design: • Lower Level earthquake with Operational with 50% probability of exceedance in 75 years (100-year return period) • Upper Level earthquake with 7% probability of exceedance in 75 years (1,000-year return period)
Criteria used in this study for Existing Bridge Retrofits	Existing WSDOT BDM-LRFD criteria for existing bridge retrofits: • 10% probability of exceedance in 50-year with 475-year return period. Existing FHWA criteria for existing bridge retrofits (same as proposed AASHTO-LRFD Criteria): • Upper Level earthquake with 7% probability of exceedance in 75 years (1,000-year return period)

3.2 Live Load Application with Seismic Loading

Sound Transit DCM uses AASHTO Standard Specifications load factors and Load Combinations that is different than the AASHTO LRFD load factors and load combinations. In this study, AASHTO LRFD Extreme Event – I load factors and load combinations are used as shown in Table 3-1.

TABLE 3-2
Load Combinations and Load Factors

Load Combination Limit State	*DC DD DW EH EV ES EL*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU CR SH*	*TG*	*SE*	Use One of These at a Time			
										EQ	*IC*	*CT*	*CV*
STRENGTH I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
FATIGUE— *LL, IM & CE* ONLY	—	0.75	—	—	—	—	—	—	—	—	—	—	—

In addition, Sound Transit DCM requires the inertia effects of the Light Rail vehicle, without impact, acting on one track only be included in the seismic analysis in deriving seismic loads.

On the other hand, AASHTO LRFD specifies that the load factor for live load in Extreme Event Load Combination I, □EQ, should be determined on a project specific basis. Also, AASHTO LRFD Commentary states that:

> *"Past editions of the Standard Specifications used □EQ = 0.0. This issue is not resolved. The possibility of partial live load, i.e., □EQ < 1.0, with earthquakes should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that □EQ = 0.50 is reasonable for a wide range of values of average daily truck traffic (ADTT)."*

Also, WSDOT BDM Section 3.1.2.B Load Factor □EQ for Live Load states that

> *"The □EQ load factor for the live load in Extreme Event-I Limit State, as specified in the LRFD Table 3.4.1-1 for the majority of bridges, shall be equal to 0.0. For bridges located in urban areas susceptible to daily heavy congestion, □EQ shall be equal to 0.5 unless otherwise directed by the WSDOT Bridge Design Engineer. The □EQ factor should be applied to the live load force effect obtained from the bridge live load analysis. Live load mass is ignored in the dynamic analysis."*

Sound Transit aerial structures design practice differs from WSDOT bridge design practice, in particular, in seismic load combination. Sound Transit includes Light Rail Vehicle mass in their seismic analysis and design. Whereas, WSDOT ignores the live load mass. When included, live load inertia effects increase the seismic loads significantly. This report is prepared by using WSDOT's current practice of using 50% of the vehicular live load in Seismic Load Combinations. In addition, 100% of the Light Rail Vehicle AW4 loading, without impact, acting on one track only is included in Seismic Load Combinations. Both vehicular and Light Rail live load included as gravity loads. Per AASHTO LRFD Extreme Event-I load combination and WSDOT's current practice, live load mass ignored in the seismic analysis.

3.3 Spectral Acceleration Values

WSDOT uses a map to identify different seismic zones with peak ground accelerations (PGA) in Washington based on USGS information. "Zone C" is considered "High Risk" and covers the area with PGA greater than 0.20 times the force of gravity. "Zone B" is considered "Moderate Risk" and contains an area of PGA between 0.10 and 0.20 times the force of gravity. "Zone A" is considered "Low Risk" and contains an area of PGA less than 0.10 times the force of gravity.

The existing bridges that were evaluated for seismic retrofits in this study are located in "Zone C" that is considered "High Risk" zone.

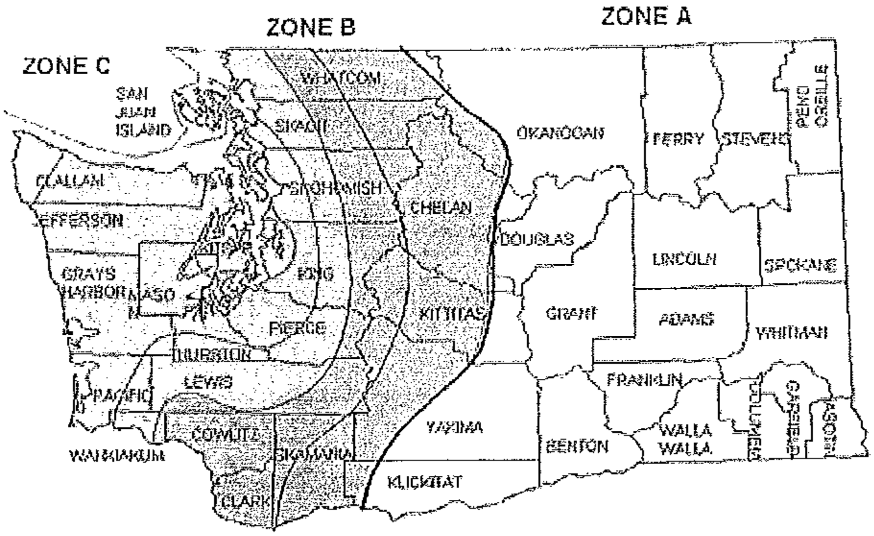


EXHIBIT 3-1

WSDOT Seismic Zone Map

This study uses USGS "Seismic Hazard Curves and Uniform Hazard Response Spectra for the United States, 2001" CD-ROM which includes a software called "Probabilistic Hazard 3.10" to create the Spectral Acceleration Values for a Site Class B (Rock) for 500, 1000 and 2500 year return periods. The 2500-year return period is used to compare and verify the spectral acceleration values that are obtained by using USGS software with the values published in Sound Transit DCM:.

Sound Transit DCM Table 8A-3 lists spectral acceleration values for MDE level design earthquake (2500-year return period). The 2500- year return period numbers we created by using the USGS software is in agreement with Table 8A-3 numbers. The 500-year numbers we created are also in agreement with standard WSDOT designs. Therefore, we believe that the 1,000-year numbers are also reasonably correct.

3.4 Elastic Seismic Response Spectrum

The current WSDOT seismic design criteria for both new bridges and retrofits are based on 2002 USGS Maps (10% probability of exceedance (PE) in 50 years, return period of 475 years). Whereas, the current Sound Transit seismic design criteria for new aerial structures is based on two level performance criteria and requires a significantly higher level of performance (Operational Earthquake with 150 years return period and Maximum Earthquake with 2500 years return period).

As owner of the existing bridges, WSDOT has indicated that they anticipate adopting a new seismic design criteria based on a 1,000-year return period as detailed in *AASHTO Guide Specifications for LRFD Seismic Bridge Design, Subcommittee for Seismic Effects on Bridges, T-3, March 2007* and FHWA Manual titled *Seismic Retrofitting Manual for Highway Structures, December 2006* instead of the current design criteria using the 475-year return period earthquake. Sound Transit is also anticipating implementing the new seismic design criteria for only future existing bridge retrofits.

A comparison of the original, 475-year and 1,000-year event response coefficients are as shown in the following figure.

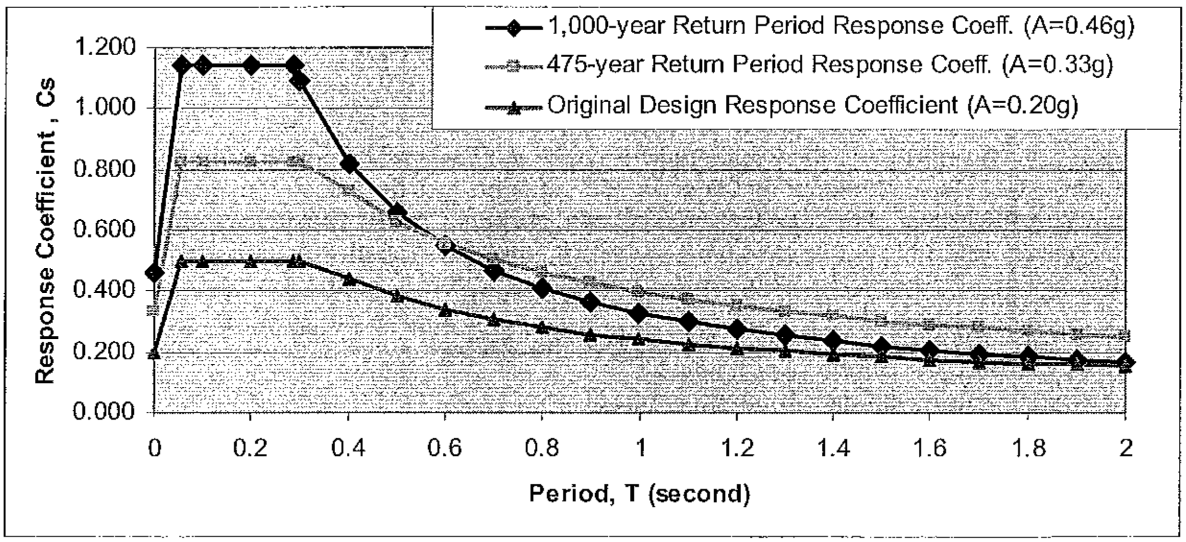


EXHIBIT 3-2
Comparison of 475-year and 1,000-year Event Coefficients

The PGA value selected as 0.33g for 475-year event based on the location of the existing bridges. A Soil Profile Type III (defined as: a profile with soft to medium-stiff clays and sands, characterized by 30 ft. or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils) and the corresponding site coefficient, S of 1.5 is used to calculate the Elastic Seismic Response Spectrum values.

AASHTO LRFD equation 3.10.6.1-1 is used to obtain, elastic seismic response coefficient, Csm, values.

$$Csm = 1.2\ A\ S\ /\ Tm^{2/3} <= 2.5A = 0.825$$

An earthquake may excite several modes of vibration in a bridge and, therefore, the elastic response coefficient should be found for each relevant mode. In this study, existing bridges are analyzed for first 80 modes of which corresponding to the period and mode shape of one of the fundamental modes of vibration. The results are combined by using root-mean-square method to obtain the effects of multiple-modes.

The PGA value calculated as 0.46g for 1,000-year event based on new criteria equations. Based on these numbers, the ratio of 475-year event to original event is about 1.65. This means 65 percent increase in the seismic demand from the original design criteria event and it is constant for all structural periods. On the other hand, the ratio of 1,000-year event to original event is about 2.30 (130 percent increase) times for short period structures and it decreases as the period of the structure increases (the ratio is about 1.20 (20 percent increase) for a period of 1.5 seconds).

Most highway bridges are expected to have at least some capacity in reserve for extreme events. The current AASHTO specifications provide varying levels of conservatism due to, among other factors, the use of relatively low R-Factors, a spectral shape based on $1/T^{2/3}$, and uncracked sections for analysis. However, the degree of conservatism is actually unknown and the consequences of earthquakes larger than the design event are uncertain and may be considerable. If the actual event is only, say 20 percent larger than the design event, damage is likely to be slight, the consequences tolerable, and the risk is acceptable. However, if the actual earthquake is 130 percent larger than the design earthquake, the reserve capacity is likely to be exceeded, and the damage is likely to be extensive.

WSDOT's Existing Bridge Retrofit Program (from Reference 3)

WSDOT has been prioritizing the bridge retrofit needs by first establishing groupings of bridges by the nature and extent of structural deficiencies. Then the bridges have been ranked according to the importance of the bridge. For superstructure retrofit ranking, Groups 1 and 2 were ranked together. For substructure retrofit ranking, Group 3 was ranked, then Group 4. Bridges are placed in one of the following groups according to their structural deficiencies.

Superstructure Group (Groups 1 and 2 were ranked together)
- Group 1: Bridges with in-span hinges.
- Group 2: Bridges simply supported at piers.

Substructure Group
- Group 3: Bridges with single-column piers.
- Group 4: Bridges with multi-column piers having substructure deficiencies.

Major/Special Bridges
- Bridges that require further structural analysis to assess whether seismic retrofit is warranted. These are essentially large or unusual type structures.

3.5.1 Program Status (as of Fall 2005)

Phase 1 (Superstructure Retrofits) – *Complete*:

Phase 2 (Single Column Retrofits and Major Bridges) – *In Progress*
- Bridges with Single Columns
  - 79 bridges completed / 10 bridges with a project in progress / 80 bridges remaining. (All these bridges should be retrofitted in 6-8 years.)

Major Bridges

15 bridges completed / 2 bridges with a project in progress / 3 bridges remaining / 3 bridges deferred / 2 bridges excluded due to future replacement planned.

Phase 3 (multiple Column Retrofits) – Future

15 bridges completed.

634 bridges remaining.

2005 Transportation Partnership Program new revenue package funded retrofit on 172 of the 634 bridges in the Puget Sound region.

3.5.2 WSDOT's Prioritization Philosophy (September 15, 2005)

- All remaining single column retrofits (Group 3) with a PGA greater than 0.15g will be placed at the top of the list.
  [East and West Approach Spans fall under this category. D2 Roadway Viaduct can be considered as single column, too]
- Next will come the remaining Major Bridges with a PGA greater than or equal to 0.3g.
  [East Channel Bridge falls under this category. D2 Roadway Viaduct – if not considered as single column bent]
- Next will come multi column bent retrofits (Group 4) in areas with PGA greater than or equal to 0.3g. The bridges identified by the legislature, for the 2005 Gas Tax, will come first, followed by the remainder ranked by ADT.
  [Rainier Avenue Bridge falls under this category. East Channel Bridge – if not considered as a Major Bridge]
- Next will come the remaining Major Bridges with a PGA greater than or equal to 0.25g.
- Next will come multi column bent retrofits (Group 4) in areas with PGA greater than or equal to 0.25g. The bridges identified by the legislature, for the 2005 Gas Tax, will come first, followed by the remainder ranked by ADT.
- Next will come multi column bent retrofits (Group 4) in areas with PGA greater than or equal to 0.2g. These will be ranked by ADT.
- Next will come the remaining bridges ranked by ADT.

As seen from the above program status and prioritization philosophy, the utmost importance was given to Superstructure Retrofits. This is because most of the pre 1980 bridges had insufficient seat lengths at the expansion joints and/or in-span hinge locations, and the superstructure retrofits are relatively simple and cheaper than substructure retrofits. Single column bents come second, and multi-column bents are last.

Chapter 4

WSDOT Provided Information

WSDOT contracted KPFF to evaluate the structural impacts of converting two lanes of the I-90 Bridges between downtown Seattle and Bellevue to Light Rail operations. The portion of the I-90 Bridge (Homer Hadley) dedicated for the Light Rail operations is the 40-foot wide LM Alignment located on the south part of the bridge.

KPFF performed a seismic screening analysis by comparing the as-built seismic criteria to the current WSDOT criteria and by comparing mass of as-built bridge to light rail transit converted bridge. WSDOT seismic screening study included only D2 Roadway Viaduct, Rainier Avenue Overcrossing (Bridge #222), and East Channel Bridge

4.1 Structural Analysis Study by KPFF

The structural evaluation included global and local analyses of the proposed LRT loading conditions, a review of Sound Transit's frequency and deflection criteria for compliance, and an assessment of changes in seismic vulnerability.

Structural Evaluation based on Group I Combinations (DL+LL):

- The global analysis is a demand vs. demand comparison of the changes in structural demand from the original to the proposed LRT design loads.
- The local analysis compares the existing capacity of superstructure elements with the demand of the proposed LRT loading.

Seismic Evaluation:

Seismic Evaluation is performed in two ways,

1. The original seismic design criteria were compared to the current WSDOT standards at the time of the study.
2. The structural mass was evaluated with a demand-demand approach. This approach is a simple comparison between the masses of the originally designed structure and the proposed LRT converted bridge. No live load is included in the mass calculations.

Seismic evaluation was performed on D2 Viaduct, East Channel Bridge and Bridge 222 (Rainier Avenue Overcrossing) only. No seismic evaluation data was provided on Transition and Approach span bridges.

4.2 WSDOT Provided Documents

The following documents were provided by WSDOT are used as references in this study:

As-Built Drawings:

- SR90 3rd Lake Washington Floating Bridge
  Approach and Transition Spans
  Alternate A2

East Channel Bridge:

As-Built Data:
Acceleration Coefficient (A) – 0.5g
Importance Classification (I) – N/A
Seismic Performance Category (SPC) – N/A
Soil Type – N/A (Assumed depth of rock 11 to 80 feet)
Site Coefficient (S) – N/A
Response Coefficient – 0.2

The original LRT track dead load is greater than the proposed LRT dead load. There is a decrease in the mass from the originally designed structure. The decrease in the mass is about 1 percent.

D2 Roadway Viaduct (Joint LRT and Bus Operations):

Same criteria as for Exclusive LRT option except the increase in the mass is 2 percent for the concrete spans and 4 percent for the steel spans due to embedded track.

Rainier Avenue Overcrossing (Bridge No. 222):

As-Built Data:
Acceleration Coefficient (A) – 0.20
Importance Classification (I) – 1 (Essential Bridge)
Seismic Performance Category (SPC) – C
Soil Type – III (Soft to medium-stiff clays and sands)
Site Coefficient (S) – 1.5
Currently the acceleration coefficient (A) is 0.29

The original LRT track dead load is greater than the proposed LRT dead load. There is a decrease in the mass from the originally designed structure. The decrease in the mass is about 2 percent.

Chapter 5

Seismic Vulnerability Analysis

For each bridge, a seismic vulnerability analysis was performed to identify critical structural components that could be damaged by a design level earthquake. The seismic vulnerability analysis includes only structural engineering aspects of the bridges. Geotechnical engineering aspects such as liquefaction is not included in this study.

The seismic risk associated with each bridge site was evaluated in terms of ground acceleration and type of foundation material. The vulnerability of the structures themselves was evaluated for adequacy of superstructure support length and apparent demand increase on the supporting elements. The structural details evaluated consisted of the following:

- Structure Type
- Bearings
- Type of Restraint
- Pier Type
- Column Type
- Column-to-Footing Anchorage Details
- Footing Type

Collapse of a superstructure is typically related to inadequate support length, which allows the superstructure to fall off its supports or failure of an in-span hinge. The demand-to-capacity evaluations for support length have been determined using the criteria specified in the FHWA's 2006 Seismic Design and Retrofit Manual for Highway Bridges.

In-span hinges are considered to be the highest seismic retrofit priority because of the potential for collapse of superstructure during relatively modest earthquakes. Most hinges have less support length than required by current AASHTO standards. Some superstructures with in-span hinges could collapse due to greater than designed movement, and others would only be susceptible to collapse if the hinge were to fail structurally; both types of deficiencies required retrofit to ensure structural safety.

Bridges that are simply supported at piers or abutments are vulnerable where the support length is inadequate and adequate restraint is not provided in either the longitudinal and transverse direction.

Bridge design criteria and structural details to accommodate dynamic seismic loading have changed dramatically over the past 20 years, based largely on lessons learned from earthquakes in California like those that occurred in 1971 at San Fernando and in 1989 at Loma Prieta. The principal areas of substructure deficiency of older bridges when compared to current design criteria are as follows:

- Inadequate confinement reinforcement for main longitudinal reinforcing steel in concrete columns
- Inadequate splice length of main longitudinal column reinforcing to footing dowels
- Inadequate development length of footing dowels (footing embedment)
- Absence of reinforcement in the tops of footings
- Inadequate footing support capacity

The first three items were easily identified by a review of the bridge plans. Splice and confinement reinforcing design and detailing practices have changed so significantly that vulnerability is more of a "YES" or "NO" determination rather than a degree of deficiency. The same is true of reinforcing in the tops of footings

In this study, for each bridge, a seismic vulnerability analysis was performed to identify critical structural components that could be damaged by a design level earthquake. We evaluated structural demand on six criteria, which are defined as follows:

- Minimum Support length demand
- Anchorage length demand for column longitudinal reinforcement
- Confinement demand for column transverse reinforcement
- Splice length demand for column longitudinal reinforcement
- Shear force demand for column
- Bending moment demand for column

The relative magnitudes of these demands may be used to sequentially upgrade a deficient bridge.

5.1 Minimum Support Lengths

The supports at the abutments, columns, and expansion joints must be of sufficient length to accommodate anticipated relative displacements. Minimum support lengths are specified because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves.

5.2 Anchorage of Longitudinal Reinforcement

A sudden loss of flexural strength can occur if longitudinal reinforcement is not adequately anchored. The pullout of longitudinal reinforcement can occur at the footings or at the bent cap. This may result either due to an inadequate anchorage length or as a result of bond degradation due to flexural or shear cracking of the concrete in the footing or cap. In either case, a sudden loss of flexural capacity may result. If inadequate anchorage length is provided for the reinforcing steel, the ultimate capacity of the steel cannot be developed and failure will occur below the ultimate moment capacity of the column.

5.2.1 Splice in Longitudinal Reinforcement

Columns that have longitudinal reinforcement spliced near or within a zone of flexural yielding may be subject to a rapid loss of flexural strength at the splice unless sufficient closely spaced transverse reinforcement is provided.

5.3 Transverse Confinement Reinforcement

Inadequate transverse confinement reinforcement in the plastic hinge region of a column will cause a rapid loss of flexural capacity due to buckling of the main reinforcement and crushing of the concrete in compression. Transverse confinement reinforcement is required to prevent strength degradation in a column subjected to reversed cycles of flexural yielding. Degradation is prevented because confinement increases the capability of the concrete core to develop significant stress at high compressive strains and prevents buckling of longitudinal compressive reinforcement by providing lateral restraint for the reinforcing bars. The degree to which degradation will be prevented is dependent on the amount and spacing of transverse reinforcing and the adequacy of the anchorage of this reinforcing.

5.4 Column Shear and Bending Moment

Column failure will occur when seismic demand exceeds the member capacity. Shear failure may occur prior to flexural yielding or during flexural yielding due to the degradation of shear capacity. Column shear failure is critical because it results in a comparatively sudden loss of shear strength. When this occurs, the resulting excessive deformations may cause disintegration of the column and the loss of vertical support.

5.5 Seismic Vulnerabilities

5.5.1 D2 Roadway Viaduct

D2 Roadway Viaduct is a twenty span structure. Spans 1 through 10 of the D2 Roadway Viaduct are continuous post-tensioned concrete box girders supported by single/multiple column bents founded on pipe pile groups. Spans 11 through 20 are continuous structural steel box girders. They are also supported by single column piers, and columns are supported by a combination of concrete piles and drilled shafts. The continuous post-tensioned superstructure has in span hinge at Spans 6.6 Piers 1, 2, 3, 6, 7, 8 and 9 have expansion bearings, while Piers 4, 5, 10 and 11 are integral to superstructure. For the continuous structural steel superstructure Piers, 12, 13, 14, 15, 16, 17, 8 and 10 have pinned bearings, while Piers 11, 18, 19, 20 and 21 have expansion bearings. Expansion joints are happening at Pier 1, in-span hinge at span 6, structural steel side of Pier 11, and 21.

From the As-Built plans, it is our understanding that this structure was designed during 1987-1989 (after AASHTO published first "Guide Specifications on Seismic Design of Highway Bridges in 1983). Therefore, consistent with the post-Seismic Guide Spec. construction practices, no column vertical bars spliced at column/footing joint region. As a result, it is expected that vertical column bars will be able to perform as designed during a seismic event.

Transverse reinforcement is #5 bars spaced at 3.5 inches for piers 1 to 10 and #6 bars spaced at 4 inches for piers 11 to 20 at the possible plastic hinge regions and they are spaced at 12 inches elsewhere. From the size and spacing of the rebar, it is expected that the columns will show ductile behavior during a seismic event.

The supports at the abutments, columns, and expansion joints must be of sufficient length to accommodate anticipated relative displacements. Minimum support lengths are specified because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves. Available seat lengths are ranging from 60 to 72 inches. These available seat lengths are not satisfying the current AASHTO requirements of 75 and 86 inches, respectively.

Numerous types of steel bearings used on various types of steel and concrete bridges have been damaged by relatively minor seismic events. In current practice, it is usually assumed that the steel bearings will fail in areas where the credible bedrock acceleration is 0.3g or greater. Also, if the failure of any type of bearing will result in dropping the superstructure 6 inches or more without falling off of the pier or abutment, the considerations should be given to replace the bearings with modern type bearings or adding bolsters that will minimize the drop. Steel rocker bearings used to support superstructure and need to be replaced.

For the spans 1 to 10 the calculated structural periods having the biggest mass participation are T_l=0.51 for the longitudinal direction and T_t=0.70 for the transverse direction. This represents about 65 percent demand increase from original design to 475-year seismic event and 72 percent from original design to 1,000-year seismic event in longitudinal direction. In the transverse direction, the demand increase between the original and 475-year seismic event is same at 65 percent. However, the demand increase between the original and 1,000-year event is about 54 percent.

For the spans 11 to 20 the calculated structural periods having the biggest mass participation are T_l=1.59 for the longitudinal direction and T_t=0.88 for the transverse direction. This represents about 65 percent demand increase from original design to 475-year seismic event and just 17 percent from original design to 1,000-year seismic event in longitudinal direction. In the transverse direction, the demand increase between the original and 475-year seismic event is same at 65 percent. However, the demand increase between the original and 1,000-year event is about 42 percent.

5.5.2 Rainier Avenue Bridge (Bridge No. 222)

Rainier Avenue Bridge is a three span post-tensioned concrete box girder bridge supported by multiple column integral bents. Column capitals have flares. Piers supported by separate concrete pile footings. The continuous post-tensioned superstructure has expansion joints at Piers 1 and 4 (abutments).

From the As-Built plans, it is our understanding that this structure was designed in 1989 (after AASHTO published first "Guide Specifications on Seismic Design of Highway Bridges in 1983). Therefore, consistent with the post-Seismic Guide Spec. construction practices, no column vertical bars spliced at column/footing joint region. As a result, it is expected that vertical column bars will be able to perform as designed during a seismic event.

Transverse reinforcement consists of 2- #9, 4-#4 and 2-#6 bars spaced at 4 inches at the possible plastic hinge regions and they are spaced at 12 inches elsewhere. From the size and spacing of the rebar, it is expected that the columns will show ductile behavior during a seismic event.

The supports at the abutments, columns, and expansion joints must be of sufficient length to accommodate anticipated relative displacements. Minimum support lengths are specified because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves. Available seat length is 30 inches at the abutments. The available seat length is satisfying the current AASHTO requirements of 29 inches.

Elastomeric bearings are used. No adverse affects of elastomeric bearings on seismic performance of bridges reported to date.

The calculated structural periods having the biggest mass participation are T_l=0.38 for the longitudinal direction and T_t=0.14 for the transverse direction. This represents about 65 percent demand increase from original design to 475-year seismic event and 85 percent from original design to 1,000-year seismic event in longitudinal direction. In the transverse direction, the demand increase between the original and 475-year seismic event is same at 65 percent. However, the demand increase between the original and 1,000-year event is about 128 percent.

5.5.3 East Channel Bridge

The East Channel Bridge is a nine span structure crossing Lake Washington's East Channel. The superstructure consists of two separate continuous structural steel box girder bridges, one carrying the eastbound (LR and LM Lines) and the other westbound (LL Line). Both bridges are sharing same multicolumn piers as their substructure. Multicolumn concrete piers are founded on spread footings. The superstructure has in span hinges at Spans 4 and 6. Piers 1, 3, 4, 7, 8 and 10 have expansion bearings, while Piers 2, 5, 6 and 9 are fixed.

From the As-Built plans, it is our understanding that this structure was built in 3 phases. In the first phase (during 1969-1970) part of Piers 3, 4, 5, 6, 7 and 8 was built. The second phase (during 1979-1983) included most of the remaining Piers and the westbound bridge. The last phase (during 1984) included eastbound piers 1 and 10, 2 columns from pier 2, 1 column from pier 9, remaining portions of the cap beams and the eastbound bridge. Therefore, consistent with the 1970's construction practices, lap splices at the bases of the columns do not have enough splice lengths (just 4'-2" for #11 bars). As a result, lap splices at the bases of the columns would almost certainly lose their flexural strength during the seismic motion. Loss of flexural strength would lead to increased damage in other parts of the structure. The splices could also fail in shear. If the splices failed in shear, the columns would not be able to support the superstructure.

Most of the columns have inadequate shear strength because their transverse reinforcement is too sparse (just #4 bars spaced at 12 inches). Shear failures must be avoided because many bridge collapses during past earthquakes (for example, the 1994 Northridge Earthquake and 1995 Kobe Earthquake) were caused by this type of failure.

The supports at the abutments, columns, and expansion joints must be of sufficient length to

accommodate anticipated relative displacements. Minimum support lengths are specified because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves. Available seat lengths are 34.5 inches at the Pier 1 and 33 inches at the Pier 10. These available seat lengths are not satisfying the current AASHTO requirements of 60 inches.

Numerous types of steel bearings used on various types of steel and concrete bridges have been damaged by relatively minor seismic events. In current practice, it is usually assumed that the steel bearings will fail in areas where the credible bedrock acceleration is 0.3g or greater. Also, if the failure of any type of bearing will result in dropping the superstructure 6 inches or more without falling off of the pier or abutment, the considerations should be given to replace the bearings with modern type bearings or adding bolsters that will minimize the drop. Steel rocker bearings used to support superstructure and needs to be replaced.

The calculated structural periods having the biggest mass participation are T_l=1.86 for the longitudinal direction and T_t=1.19 for the transverse direction. This represents about 34 percent demand increase from original design to 475-year seismic event and no increase from original design to 1,000-year seismic event in longitudinal direction. In the transverse direction, the demand increase between the original and 475-year seismic event is same at 76 percent. However, the demand increase between the original and 1,000-year event is about 36 percent.

5.5.4 West Approach Spans

West Approach Span Bridge is a six span segmental concrete box girder bridge supported by single column integral bents. Column capitals have flares. Piers supported by separate concrete pile footings. The continuous post-tensioned superstructure has an expansion joint at Pier 1 (abutment).

From the As-Built plans, it is our understanding that this structure was designed during 1984-1987 (after AASHTO published first "Guide Specifications on Seismic Design of Highway Bridges in 1983). Therefore, consistent with the post-Seismic Guide Spec. construction practices, no column vertical bars spliced at column/footing joint region. As a result, it is expected that vertical column bars will be able to perform as designed during a seismic event.

Transverse reinforcement consists of 22- #4 and 1-#6 bars spaced at 3.5 inches at the possible plastic hinge regions and they are spaced at 12 inches elsewhere. From the size and spacing of the rebar, it is expected that the columns will show ductile behavior during a seismic event.

The supports at the abutments, columns, and expansion joints must be of sufficient length to

Accommodate anticipated relative displacements. Minimum support lengths are specified because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves. Available seat length is 55 inches at the abutment. The available seat length is not satisfying the current AASHTO requirements of 59 inches.

Piers 3 to 7 have integral connection with the superstructure. Elastomeric bearings are used on Piers 1 and 2. No adverse affects of elastomeric bearings on seismic performance of bridges reported to date.

The calculated structural periods having the biggest mass participation are T_l=0.41 for the longitudinal direction and T_t=0.69 for the transverse direction. This represents about 65 percent demand increase from original design to 475-year seismic event and 85 percent from original design to 1,000-year seismic event in longitudinal direction. In the transverse direction, the demand increase between the original and 475-year seismic event is same at 65 percent. However, the demand increase between the original and 1,000-year event is about 54 percent.

5.5.5 East Approach Spans

East Approach Span Bridge is a seven span segmental concrete box girder bridge supported by single column integral bents. Column capitals have flares. Piers supported by separate concrete pile footings. The continuous post-tensioned superstructure has an expansion joint at Pier 16 (abutment).

From the As-Built plans, it is our understanding that this structure was designed during 1984-1985 (after AASHTO published first "Guide Specifications on Seismic Design of Highway Bridges in 1983). Therefore, consistent with the post-Seismic Guide Spec. construction practices, no column vertical bars spliced at column/footing joint region. As a result, it is expected that vertical column bars will be able to perform as designed during a seismic event.

Transverse reinforcement of columns at Piers 9, 10 and 11 spaced at 8 inches, whereas the transverse reinforcement of columns at Piers 12 to 15 spaced at 4 inches at the possible plastic hinge regions. From the size and spacing of the rebar, it is expected that the columns at the Piers 12 to 15 will show ductile behavior during a seismic event. Whereas, columns at the Piers 9 to 11 may not be able to perform as good as rest of the columns.

The supports at the abutments, columns, and expansion joints must be of sufficient length to

Accommodate anticipated relative displacements. Minimum support lengths are specified because an elastic analysis does not account for the effects of nonlinear response of the structure or variation in motions at the support due to traveling surface waves. Available seat length is 55 inches at the abutment. The available seat length is not satisfying the current AASHTO requirements of 64 inches.

Piers 9 to 14 have integral connection with the superstructure. Elastomeric bearings are used on Piers 15 and 16. No adverse affects of elastomeric bearings on seismic performance of bridges reported to date.

The calculated structural periods having the biggest mass participation are T_l=0.33 for the longitudinal direction and T_t=0.68 for the transverse direction. This represents about 65 percent demand increase from original design to 475-year seismic event and 99 percent from original design to 1,000-year seismic event in longitudinal direction. In the transverse direction, the demand increase between the original and 475-year seismic event is same at 65 percent. However, the demand increase between the original and 1,000-year event is about 55 percent.

5.5.6 Transition Spans

Our evaluation indicated that the transition spans are not vulnerable to a design-level earthquake since they are simple span bridges.

TABLE 5-1
D2 Roadway Viaduct
(LRT Only & Joint LRT and Bus Operations Alternatives)

Seismic Deficiency	Vulnerability *(High or Low)*	Comments
Bearing Seat Length or Expansion Joint	**High**	Expansion joints are happening at Pier 1 (abutment), in-span hinge at span 6, structural steel side of Pier 11, and Pier 21 (abutment). Available seat lengths at these expansion joints are not satisfying the current FHWA Seismic Retrofitting Manual for Highway Structures requirements.
Type of Bearings	**High**	Steel rocker bearings used to support structural steel box girder superstructure (Spans 11 through 20). In current practice, it is usually assumed that the steel bearings will fail in areas where the credible bedrock acceleration is 0.3g or greater.
Column Longitudinal Reinforcement Anchorage Length	Low	It appears that column longitudinal reinforcement anchored properly.
Column Transverse Reinforcement Confinement	Low	Transverse reinforcement is #5 bars spaced at 3.5 inches for piers 1 to 10 and #6 bars spaced at 4 inches for piers 11 to 20 at the possible plastic hinge regions and they are spaced at 12 inches elsewhere. From the size and spacing of reinforcement it is expected that the columns will show some ductile behavior during a seismic event. Push over analysis needs to be performed to determine the extent of expected ductile behavior.
Column Longitudinal Reinforcement Splice Length	Low	As-built plans do not show any longitudinal reinforcement splices at the possible plastic hinge regions.
Column Shear Force and Bending Moment	**High**	There is significant seismic load demand increase due to significant changes in the seismic criteria. Refined analysis needs to be performed to determine if the seismic retrofit is required.
Footings	Low	Provided reinforcement detailing is comparable to modern current seismic detailing practice.
Liquefaction Potential		Not evaluated as part of this study.

TABLE 5-2
Rainier Avenue Overcrossing
(Bridge No. 222)

Seismic Deficiency	Vulnerability *(High or Low)*	Comments
Bearing Seat Length or Expansion Joint	High	Expansion joints are happening at Piers 1 and 4 (abutments). Available seat lengths at these expansion joints are not satisfying the current FHWA Seismic Retrofitting Manual for Highway Structures requirements.
Type of Bearings	Low	Elastomeric bearings used to support superstructure. No adverse affects of elastomeric bearings on seismic performance of bridges were reported to date.
Column Longitudinal Reinforcement Anchorage Length	Low	It appears that column longitudinal reinforcement anchored properly.
Column Transverse Reinforcement Confinement	Low	Transverse reinforcement consists of 2- #9, 4-#4 and 2-#6 bars spaced at 4 inches at the possible plastic hinge regions and they are spaced at 12 inches elsewhere. From the size and spacing of reinforcement it is expected that the columns will show some ductile behavior during a seismic event. Push over analysis needs to be performed to determine the extent of expected ductile behavior.
Column Longitudinal Reinforcement Splice Length	Low	As-built plans do not show any longitudinal reinforcement splices at the possible plastic hinge regions.
Column Shear Force and Bending Moment	High	There is significant seismic load demand increase due to significant changes in the seismic criteria. Refined analysis needs to be performed to determine if the seismic retrofit is required.
Footings	Low	Provided reinforcement detailing is comparable to modern current seismic detailing practice.
Liquefaction Potential		Not evaluated as part of this study.

TABLE 5-3
East Channel Bridge

Seismic Deficiency	Vulnerability *(High or Low)*	Comments
Bearing Seat Length or Expansion Joint	**High**	Expansion joints are happening at Pier 1 (abutment), in-span hinge at spans 4 and 6, and at Pier 11 (abutment). Available seat lengths at these expansion joints are not satisfying the current FHWA Seismic Retrofitting Manual for Highway Structures requirements.
Type of Bearings	**High**	Steel rocker bearings used to support structural steel box girder superstructure (Spans 11 through 20). In current practice, it is usually assumed that the steel bearings will fail in areas where the credible bedrock acceleration is 0.3g or greater.
Column Longitudinal Reinforcement Anchorage Length	**High**	It appears that column longitudinal reinforcement does not anchored properly.
Column Transverse Reinforcement Confinement	**High**	Most of the columns have inadequate shear strength because their transverse reinforcement is too sparse (just #4 bars spaced at 12 inches). From the size and spacing of reinforcement it is expected that the columns will not show ductile behavior during a seismic event. Push over analysis needs to be performed to determine the extent of expected ductile behavior.
Column Longitudinal Reinforcement Splice Length	**High**	The columns for the Piers 3, 4, 5, 6, 7 and 8 which were built in the first Phase do not have enough lap splice lengths (just 4'-2" for #11 bars). As a result, lap splices at the bases of the columns would almost certainly lose their flexural strength during the seismic motion.
Column Shear Force and Bending Moment	**High**	There is some moderate seismic load demand increase. Refined analysis needs to be performed by considering seismic isolation of the superstructure to determine if the seismic retrofit is required for the substructure.
Footings	**High**	The footings for the Piers 3, 4, 5, 6, 7 and 8 which were built in the first Phase do not have top reinforcement mat.
Liquefaction Potential		Not evaluated as part of this study.

TABLE 5-4
West Approach Spans

Seismic Deficiency	Vulnerability *(High or Low)*	Comments
Bearing Seat Length or Expansion Joint	High	Expansion joint is at Pier 1 (abutments). Available seat length at the expansion joint is not satisfying the current FHWA Seismic Retrofitting Manual for Highway Structures requirements.
Type of Bearings	Low	Elastomeric bearings used to support superstructure. No adverse affects of elastomeric bearings on seismic performance of bridges were reported to date.
Column Longitudinal Reinforcement Anchorage Length	Low	It appears that column longitudinal reinforcement anchored properly.
Column Transverse Reinforcement Confinement	Low	Transverse reinforcement consists of 22- #4 and 1-#6 bars spaced at 3.5 inches at the possible plastic hinge regions and they are spaced at 12 inches elsewhere. From the size and spacing of reinforcement it is expected that the columns will show some ductile behavior during a seismic event. Push over analysis needs to be performed to determine the extent of expected ductile behavior.
Column Longitudinal Reinforcement Splice Length	Low	As-built plans do not show any longitudinal reinforcement splices at the possible plastic hinge regions.
Column Shear Force and Bending Moment	High	There is significant seismic load demand increase due to significant changes in the seismic criteria. Refined analysis needs to be performed to determine if the seismic retrofit is required.
Footings	Low	Provided reinforcement detailing is comparable to modern current seismic detailing practice.
Liquefaction Potential		Not evaluated as part of this study.

TABLE 5-5
East Approach Spans

Seismic Deficiency	Vulnerability (High or Low)	Comments
Bearing Seat Length or Expansion Joint	**High**	Expansion joint is at Pier 16 (abutment). Available seat length at the expansion joint is not satisfying the current FHWA Seismic Retrofitting Manual for Highway Structures requirements.
Type of Bearings	**Low**	Elastomeric bearings used to support superstructure. No adverse affects of elastomeric bearings on seismic performance of bridges were reported to date.
Column Longitudinal Reinforcement Anchorage Length	**Low**	It appears that column longitudinal reinforcement anchored properly.
Column Transverse Reinforcement Confinement	**Low**	Transverse reinforcement of columns at Piers 9, 10 and 11 spaced at 8 inches, whereas the transverse reinforcement of columns at Piers 12 to 15 spaced at 4 inches at the possible plastic hinge regions. From the size and spacing of the rebars, it is expected that the columns at the Piers 12 to 15 will show ductile behavior during a seismic event. Whereas, columns at the Piers 9 to 11 may not be able to perform as good as rest of the columns. Push over analysis needs to be performed to determine the extent of expected ductile behavior.
Column Longitudinal Reinforcement Splice Length	**Low**	As-built plans do not show any longitudinal reinforcement splices at the possible plastic hinge regions.
Column Shear Force and Bending Moment	**High**	There is significant seismic load demand increase due to significant changes in the seismic criteria. Refined analysis needs to be performed to determine if the seismic retrofit is required.
Footings	**Low**	Provided reinforcement detailing is comparable to modern current seismic detailing practice.
Liquefaction Potential		Not evaluated as part of this study.

Chapter 6

Conclusion

Based on design team's seismic evaluation study on the existing bridges, this report concludes that:

- The effect of the Live Load increase on substructure elements due to Light Rail Vehicle is negligible and does not create a seismic load demand increase from the original design.
- Since the Light Rail Transit related dead loads are within plus/minus couple of percentage points of the as-built plan dead load provisions for the existing bridges, there are no seismic demand increase due to Light Rail Transit related structural modifications.
- Since there are significant changes in the seismic design criteria and detailing practices during the last 30 years, Sound Transit and WSDOT need to determine whether to retrofit the existing I-90 bridges that will be utilized by the Light Rail Vehicles to the current criteria. If the new criteria is utilized, there is significant seismic demand increase on the existing bridges.
- Refined methods of evaluation such as Pushover Analysis should be performed to investigate existing structure's ability to accommodate expected seismic displacements.

APPENDIX A

WSDOT Letter

Eastside HCT Corridor
Existing I-90 Bridges
Vibration Study
Final Conceptual Report

Prepared for:
Sound Transit

Prepared by:
Sound Transit East Link Project Team

January 2008

Quality Tracking:

Resource Lead: Ahmet Ozkan, P.E.. (AO)	**Date Submitted for review:** (Date)
Senior Reviewer: Daniel P. Russell, P.E. (DPR)	**Review completion date:** (Date)
Task Leader Verified with originator: (Name/ wet initials)	**Date revised/initials:** (date and originator's initials)
Editor: (Name/wet initials)	**Date finalized for client review:** (Date)

EAST LINK PROJECT

Existing I-90 Bridges Vibration Study Final Conceptual Report

January 2008

SOUNDTRANSIT

CENTRAL PUGET SOUND REGIONAL TRANSIT AUTHORITY

SOUND TRANSIT

SOUND TRANSIT EAST LINK PROJECT
Phase 2

Existing I-90 Bridges Vibration Study Final Conceptual Report

Prepared for:
CH2M HILL, Inc. and Sound Transit

Prepared by:
INCA Engineers, Inc.

January 2008

Translation services and information in accessible formats are available upon request by calling 1.800.201.4900 (voice) or 206.398.5410 (TTY).

For more information about the Vibration Study Final Conceptual Report call (refer to specific community relations coordinator as appropriate) or write Sound Transit, 401 South Jackson Street, Seattle, WA 98104-2826. You may also e-mail Sound Transit at main@soundtransit.org, visit our Web site at www.soundtransit.org or call our toll free information line at 1-800-201-4900.

Certificate of Engineer

The work contained herein was prepared under the supervision and direction of the undersigned

EXPIRES 10/5/

Ahmet Ozkan
Chief Bridge Engineer

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Chapter 1

Executive Summary

This report describes results of an independent structural dynamic analyses performed on the existing I-90 bridges between downtown Seattle and Bellevue to evaluate the effects of Sound Transit's "Vibration Control" and "Deflection Control" criteria. WSDOT retained KPFF to perform a similar analysis on these existing structures in the past. In their evaluation of "Vibration Control" criteria, KPFF included both simple and continuous span bridges. However, simple span bridges are much more susceptible to adverse effects of vibration than continuous span bridges. Therefore, the Sound Transit Design Criteria Manual has provisions for only simple span bridges for "Vibration Control".

Our results confirmed WSDOT's findings on both frequency and deflection checks based on Sound Transit design manual criteria as follows:

- Transition Spans: Failed to satisfy both "Vibration Control" and "Deflection Control" criteria.
- West Approach Structure: Satisfied "Deflection Control" criteria.
- East Approach Structure: Satisfied "Deflection" criteria.
- D2 Viaduct – Concrete Spans: Satisfied "Deflection" criteria.
- D2 Viaduct – Steel Spans: Satisfied "Deflection" criteria
- Rainier Avenue Overcrossing (Bridge No. 222): Satisfied "Deflection Control" criteria.
- East Channel Bridge: Satisfied "Deflection" criteria.

The Sound Transit design criteria manual has provisions to include a nominal impact factor of 30 percent of the light rail vehicle live load to account for dynamic interactions between the aerial structures and the light rail vehicle. For the Transition Spans, the only bridges that failed to satisfy the "Vibration Control" criteria, impact factors were calculated to evaluate whether the impact loads in excess of the design criteria manual minimum value (30 percent of live load) are required. Our analysis indicates that the impact factors are less than 30 percent for the Transition Spans when the light rail vehicles are operated at speeds of 55 mph or less. The impact factors increase as the light rail vehicle speed increases. Therefore, the light rail vehicle operating at speeds higher than 55 mph would require the use of increased impact factors and potential remediation for this increased loading. Since we anticipate that the vehicle speeds will be maintained at or below 55 mph, our analyses used the Sound Transit required minimum impact factor of 30 percent for the Transition Span's evaluations.

For the Transition Spans, the only bridges that did not meet the Sound Transit "Deflection Control" criteria, 3-D finite element models were created for further study. These models were used to study the acceleration levels due to light rail passage at different speeds in order to evaluate the rider comfort and determine if the accelerations reach levels that significantly degrade the ride quality. We found that at an operating speed of 55 mph, the accelerations approached about 2.0 percent of gravitational acceleration, which is less than the threshold value of 5.0 percent of gravitational acceleration per ISO 2631.

Based on our findings, at this conceptual stage, no additional remedial action is required on the existing I-90 bridges to satisfy Sound Transit's "Vibration Control" and "Deflection Control" design criteria requirements, for train speeds less than 55 mph.

In this study, the consultant team used the Empirical Impact Factors for "Vibration Control" evaluations, and Moving Load Analysis for "Deflection Control" evaluations. As the Eastside HCT Project advances into the preliminary and final design phases, the consultant team will be re-evaluating the vehicle-bridge interaction in more detail. If required, more refined analysis models will be created to investigate the Vehicle-Structure Interaction Dynamic Analysis in order to insure the structural integrity and comfort of the riders. Mitigation measures will be implemented if the analyses determines that it is necessary.

Chapter 2

Introduction

This report will analyze the existing I-90 bridges between downtown Seattle and Bellevue to determine the ride comfort and the bridge structural integrity when Light Rail is added. The concept of riding comfort is related to physical quantities such as vibration and acceleration that occur when the light rail vehicle travels over the supporting structure. The structural integrity of the bridge is related to vibration and any amplification as the train crosses the structure. There are various means by which the magnitude of vibration can be expressed, such as displacement, velocity and acceleration. The Sound Transit Design Criteria Manual requires "Deflection Control" to ensure rider comfort. Under normal live load, it is required that deflection of longitudinal girders should not exceed more than 1/1000 of the span length. In addition to the "Deflection Control", Sound Transit has "Vibration Control" provisions to limit vibrational amplification and determine whether the impact loads in excess of 30 percent of light rail vehicle live load are required for the design of the aerial structures.

Recently, WSDOT evaluated the structural impacts of converting two lanes of the I-90 Bridges between Seattle and Bellevue to Light Rail Transit operations. The WSDOT study included the following structures:

- Transition Spans
- Approach Structures
- D2 Roadway Viaduct
- Rainier Avenue Overcrossing (Bridge #222)
- East Channel Bridge

The WSDOT study identified the structures that failed to satisfy the Sound Transit criteria as the Transition Spans, the approach structures, the D2 Viaduct, and the East Channel Bridge for "Vibration Control" and the Transition Spans for "Deflection Control" provisions. However, based on Sound Transit's Design Criteria Manual, the "Vibration Control" criteria are applicable to only the simple span bridges. Therefore, in this study, the continuous span bridges, (Approach spans, D2 Roadway Viaduct, East Channel Bridge and Rainier Avenue Overcrossing), were not evaluated for "Vibration Control" requirements.

The objective of this report is to perform independent structural analyses to verify the WSDOT provided information; identify the structure(s) that fail to satisfy either "Deflection Control" or "Vibration Control" or both criteria; if applicable, perform more detailed structural analysis on bridges that failed to satisfy the criteria; develop mitigation measures, if needed; and develop the quantities for cost estimating purposes

2.1 REFERENCES

The following publications are referenced in this report.

1. ACI 358.1R-92, Analysis and Design of Reinforced and Prestressed-Concrete Guide Way Structures, ACI Manual of Concrete Practice 2005, Part 5
2. Y.B. Yang, J.D. Yau, Y.S. Wu, Vehicle-Bridge Interaction Dynamics, with Applications to High-Speed Railways, World Scientific, 2004
3. L. Fryba, Dynamics of Railway Bridges, Thomas Telford, 1996

Chapter 3

Sound Transit Criteria

Sound Transit Design Criteria Manual Section 8.5.4 Special Design Considerations have the following provisions for the vibration and deflection control of the aerial structures.

> <u>Vibration Control:</u> *"To limit vibrational amplification due to the dynamic interaction between the superstructure and the rail car, the first-mode natural frequency of flexural vibration of each simple span guide way should generally be not less than 2.5 hertz and no more than one span in a series of three consecutive spans should have a first-mode natural frequency of less than 3.0 hertz. Long simple spans having lower natural frequencies may be used, provided that due consideration is given to possible vibrational interactions between the structure and the rail car, and their effect on vertical impact loading. A special analysis shall be conducted for any bridge or superstructures having a first mode of vertical vibration, which is less than 2.5 hertz, or for the condition when more than one span in a series of three consecutive spans has the first mode of vibration which is less than 3.0 hertz. This analysis shall model the proposed structure and the transit vehicle. The analysis shall contain a sufficient number of degrees of freedom to allow modeling of the structure, vehicle truck spacing, vehicle primary suspension, vehicle secondary suspension, and the car body. It shall make provision for the placement of the vehicle on the structure in various locations in order to model the passage of the transit vehicle. When the exact configuration of either the vehicle or the structure is not known, the study shall assume a reasonable range of parameters and shall model combinations of those parameters as deemed appropriate. The analysis shall determine whether impact loads in excess of 30 percent of rail transit are required for the design of the structure. The analysis shall also determine whether certain operational considerations such as speed restriction or other provisions are required in order to ensure the safe operation of the rail transit over the structure."*

> <u>Deflection Control:</u> *"To ensure rider comfort, the deflection of longitudinal girders under normal live load should not exceed 1/1000 of the span length. For main cantilever girders, the deflection under normal live load should not exceed 1/375 of the cantilever span."*

Based on the above Sound Transit criteria the "Vibration Control" limits were imposed to limit vibrational amplification due to dynamic interaction between the superstructure and rail car and to determine whether impact loads in excess of design criteria manual value of 30 percent of live load are required for the design of the simple span aerial structures. Whereas, the "Deflection Control" limits were imposed to ensure rider comfort.

Chapter 4

WSDOT Provided Information

WSDOT retained KPFF to evaluate the structural impacts of converting two lanes of the I-90 Bridges between downtown Seattle and Bellevue to Light Rail operations. The portion of the I-90 Bridge (Homer Hadley) dedicated for the Light Rail operations is the 40-foot wide LM Alignment located on the south part of the bridge. The WSDOT study included the Transition Spans, Approach Structures, D2 Roadway Viaduct, Rainier Avenue Overcrossing (Bridge #222), and East Channel Bridge. The bridges were evaluated for compliance with Sound Transit's frequency and deflection criteria.

The following documents were provided by WSDOT are used as references in this study.

As-Built Drawings:

- SR90 3rd Lake Washington Floating Bridge
  Approach and Transition Spans
  Alternate A2
- SR90 Seattle Transit Access
  Concrete Alternative
  Transit D-2
- SR90 Seattle Transit Access
  Steel Alternative
  Transit D-2
- SR90 Bush Place to 23rd Avenue South
  Eastbound and Center Roadway
  Bridge No. 222
- SR90 Seattle Transit Access
  Concrete Alternative
  Transit D-2

Study Reports:

- Homer Hadley (Interstate 90) Floating Bridge
  Approach Structure and Transition Span
  Draft Structural Analysis Study for Light Rail Conversion
  By KPFF Consulting Engineers, August 31, 2001
- I-90 Light Rail Transit Usage Conversion Study
  Draft Structural Analysis Study for
  D2 Roadway Viaduct
  Rainier Avenue Overcrossing and
  East Channel Bridge
  By KPFF Consulting Engineers, September 15, 2006

A brief description of each bridge is as follows:

<u>Transition Spans:</u>

The Transition Spans located between the land-founded approach bridges and the floating bridge along the LM line that support a 40-foot roadway. Each Transition Span is a simple span bridge consists of two structural steel box girders composite with 8.25-inch concrete deck. The centerline bearing to centerline bearing span length of the Transition Spans are 187 and 197 feet for the west and east spans, respectively. The plate thicknesses changes four times to provide increased stiffness towards the mid span. The west Transition Span is located between Stations 1135+60.00 and 1137+60.00. The east Transition Span is located between Stations 1194+90.00 and 1196+90.00.

Approach Bridges:

The east and west approach structures along the LM line are post-tensioned concrete segmental bridges that support a 40-foot wide roadway. The deck is approximately 42-feet wide and cantilevers out 9-feet on each side of sloped webs. The cross section of the box girder has varying depth, increasing in depth near the piers. The 1,270-foot east approach bridge consists of seven continuous spans with a maximum span of 235 feet. The 1100-foot west approach bridge consists of six continuous spans with a maximum span of 262 feet. These bridges are located between Stations 1122+05.66 and 1209+58.56.

D2 Roadway Viaduct:

The D2 roadway viaduct is a 20 span structure located between Station 1005+41.96 and Station 1035+09.43. From Pier No's.1 to 11, the structure is a post-tensioned concrete box girder bridge and from Pier No's.11 to 21, it is a structural steel box girder bridge.

Rainier Avenue Overcrossing (Bridge No. 222):

The Bridge No.222 is a three span multi-cell post-tensioned concrete box girder bridge. The structure is located between the Stations 1073+80.00 and 1076+85.00 along the LM line.

East Channel Bridge:

The East Channel Bridge is a nine span structure beginning at Station 1346+01.85 and terminating at Station 1386+25.95. The superstructure consists of multiple structural steel box girders composite with concrete deck.

A summary of the WSDOT provided calculation results pertaining to the frequencies and deflections of the bridges are displayed in the Table 4-1.

Note that the WSDOT provided calculations, performed by KPFF, assumed that the Sound Transit "Vibration Control" provisions related to both single and continuous spans. Based on that interpretation, KPFF performed a frequency analysis on all the bridges regardless of whether they had simple or continuous spans.

TABLE 4-1
KPFF Calculated Frequencies and Deflections

Description	Frequency Values *(Hz)*	Frequency Check *(OK / Not OK)*	Deflection Values *(inch)*	Deflection Values *(d/L)*	Deflection Check *(OK / Not OK)*
Transition Spans *(Simple Span Bridge)*					
	2.35	Not OK	4.0	1/597	Not OK
Approach Spans *(Continuous Spans)*					
West Approach Structure	1.6 to 3.5	N/A	0.12 to 1.56	1/8820 to 1/2015	OK
East Approach Structure	2.7 to 4.0	N/A	0.12 to 1.08	1/11150 to 1/2611	OK
D2 Roadway Viaduct (Exclusive LRT) *(Continuous Spans)*					
Concrete Spans	0.96 to 1.88	N/A	0.20 to 0.60	1/6660 to 1/3100	OK
Steel Spans	1.21 to 2.13	N/A	0.3 to 1.5	1/3560 to 1/1504	OK
D2 Roadway Viaduct (Joint LRT/Bus Operations) *(Continuous Spans)*					
Concrete Spans	0.96 to 1.88	N/A	0.20 to 0.60	1/6660 to 1/3100	OK
Steel Spans	1.15 to 2.08	N/A	0.3 to 1.5	1/3560 to 1/1504	OK
Rainer Avenue Overcrossing (Bridge No. 222) *(Continuous Spans)*					
	3.3 to 5.0	N/A	0.20 to 0.30	1/5160 to 1/4760	OK
East Channel Bridge *(Continuous Spans)*					
	1.7	N/A	0.7 to 3.1	1/3154 to 1/1161	OK

From Table 4-1, KPFF's initial study concluded that except for the Rainier Avenue Overcrossing (Bridge No. 222), all the bridges failed to satisfy the Sound Transit's "Vibration Control" criteria. However, since Sound Transit's "Vibration Control" criteria is only applicable to simple spans, only the Transition Spans need to satisfy the "Vibration Control" criteria.

Table 4-1 also shows that, except for the Transition Spans, all of the bridges satisfied the Sound Transit's "Deflection Control" criteria. Therefore, only the Transition Spans require further evaluation to ensure Sound Transit's rider comfort provisions are met.

Chapter 5

Structural Analysis

For each bridge, an independent structural analysis was performed to verify the WSDOT provided information. The 3-D Finite Element Models with different degrees of sophistication created to accurately capture the dynamic characteristics of each bridge. For example, the Transition Spans are simple span bridges, but one end of each span is supported by the pontoons. To be able to capture the dynamic characteristics accurately, boundary conditions included spring supports to mimic the vertical motion of the pontoons through the water and also included 50 percent added hydraulic mass. The finite element analysis models were created using GTSTRUDL software. In addition, LUSAS and Solid Works/COSMOS software were used when higher levels of analysis were needed.

5.1 Vibration Control Analysis Results

Currently, Sound Transit has established a "Vibration Control" criteria based on first natural frequency of the unloaded structure to ensure that the natural frequency of the aerial structures do not go below a certain limit. This lower limit usually depends on the type of the aerial structures, but traditionally has been set as 2.5 Hz. Although the 2.5 Hz limit is cited in many of the transit agencies design criteria manuals, including Sound Transit's, the origin of this frequency limit is unknown. Most probably, this limit was set to avoid dynamic amplification caused by vehicles traveling at high speeds (over 100 mph) on short successive simple spans (less than 100 feet) since they would effectively be launched from one span to next in resonance with some vibration mode, causing amplified dynamic effects. Since the natural frequency of the vehicle suspension systems are typically below 2.5 Hz, a viable method to minimize the resonance effect was to have the vehicle and aerial structure frequencies as far apart as practicable. This approach is valid for typical short span structures since the natural frequencies higher than 2.5 Hz can easily and practically be achieved. But, for longer spans, it is almost impossible to satisfy this limit without substantially increasing the cost of the structure.

In design practice, the dynamic response of a bridge has been indirectly considered by increasing the forces and stresses caused by the static live loads by an "Impact Factor", defined as the ratio of the maximum dynamic to maximum static response of the bridge under the same load. The Sound Transit criteria suggests that when the natural frequency of the unloaded bridge is less than 2.5 Hz, the dynamic effects can reach significant values and must be considered.

The vibration control analysis involves creating the structural and vehicle models as accurate as possible in respect to their geometry, mass, stiffness and boundary conditions. The models available in practice for consideration of dynamic effects are, in terms of increasing complexity:

- Empirical Impact Factors
- Moving Load Analysis
- Vehicle-Structure Interaction Dynamic Analysis

There are, basically, two effects that are associated with the motion of the vehicle over a bridge: the gravitational effect and the inertia effect, both related to the mass of the vehicle. For the cases where the mass of the vehicle is small compared to mass of the bridge, the vehicle can be represented by neglecting the inertia effects. Based on the Reference 3 (L. Fryba, Dynamics of Railway Bridges, Thomas Telford, 1996), *"if the inertia effects of moving vehicles are considerably less than their weight effects, the inertia forces can be entirely neglected. This can be permitted in the case of medium and large span bridges (over 30 m) [where] the self-weight of which is considerably higher than the vehicle weight".*

In our case, the length of Transition Span is 197 feet and the weight ratio is a little less than 10 percent of the superstructure for a single car and less than 20 percent for a fully-load span with two cars (Transition Span weight is about 1,600 kips and Light Rail Vehicle weight is about 150 kips, which give ratios of 150/1,600 = 0.094 for a single car and 300/1,600= 0.188 for two cars). Therefore, at this conceptual design stage, our analysis has not included the suspension system of the vehicle in the analysis (no Vehicle-Structure Interaction Dynamic Analysis was performed). When included the suspension system of the vehicle will further dampen the load imported to the structure. In this study, the consultant team used the Empirical Impact Factors for "Vibration Control" evaluations, and Moving Load Analysis for "Deflection Control" evaluations.

One empirical method used to calculate the amount of dynamic amplification is given in 358.1R-92 (Reference 1), which defines the impact load (dynamic load allowance) as directly proportional to the vehicle crossing frequency (VCF = speed/span), and inversely proportional to the aerial structure natural frequency (f_1) as shown in Table 5-1. The Transition Spans were evaluated based on the equations in Table 5-1 to see if any increase in the impact factors were warranted and the results are shown in Tables 5-2 though 5-3.

TABLE 5-1
ACI Impact Factors

Table 3.3.1.2 Dynamic Load Allowance (Impact)

Structure Types	Rubber-tired and Continuously Welded Rail	Jointed rail
Simple-span structures, $I = \frac{VCF}{f_1} - 0.1$	≥0.10	≥0.30
Continuous-span structures, $I = \frac{VCF}{2f_1} - 0.1$	≥0.10	≥0.30

Recently, Yang et. al. (Reference 2) studied the impact response of simple beams subjected to moving train loads and created upper bound envelopes for the midpoint deflections, bending moments, and near-support shear forces, which all related to the nondimensional speed parameter (S_1), defined as the ratio of exciting frequency of the moving load to the fundamental frequency of the beam, Figures 5-1, 5-2 and 5-3. In addition to the ACI method, the Transition Spans were also evaluated based on these upper bound equations as shown in Figures 5-1 to 5-3 (from Yang et. al.), (where vt is the vehicle speed (ft/sec) times time (sec) and L (ft) is the span length. The ratio of vt/L gives the acting position of the moving load along the span lengths i.e., vt/L = 4/8 means, mid span) to see if any increase in the impact factor was warranted (Tables 5-2 through 5-3).

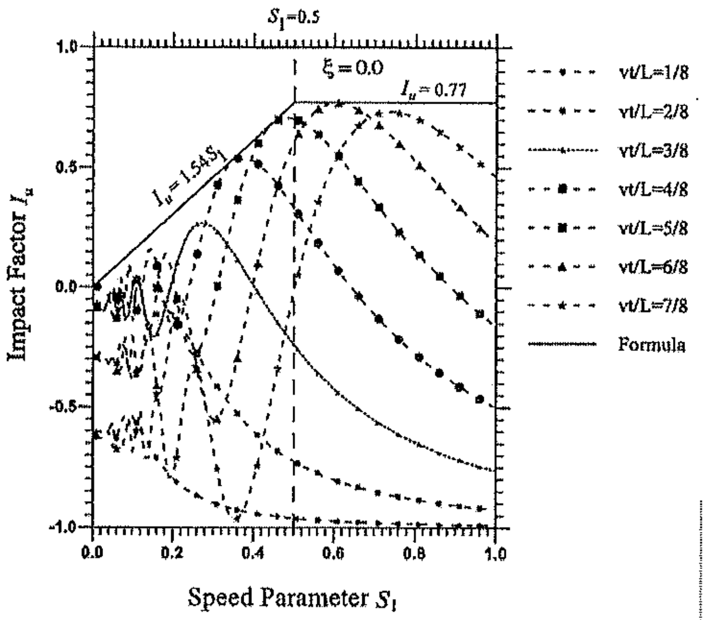


EXHIBIT 5-1
Impact Factors for Deflection

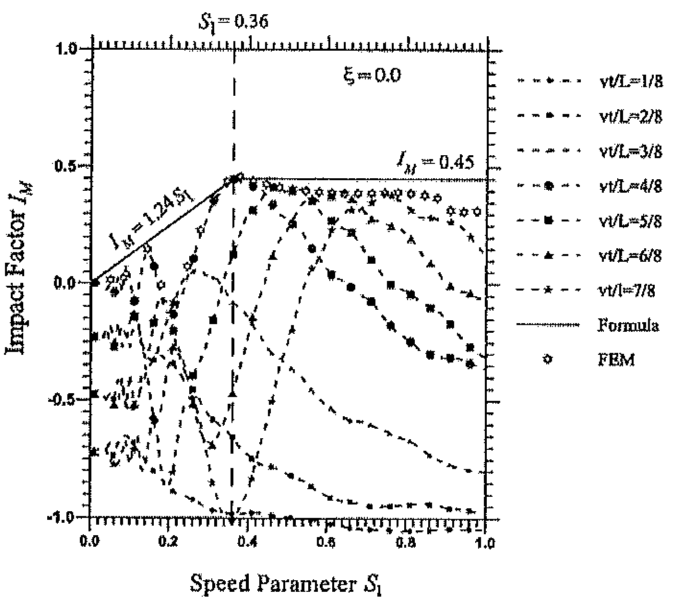


EXHIBIT 5-2
Impact Factors for Bending Moment

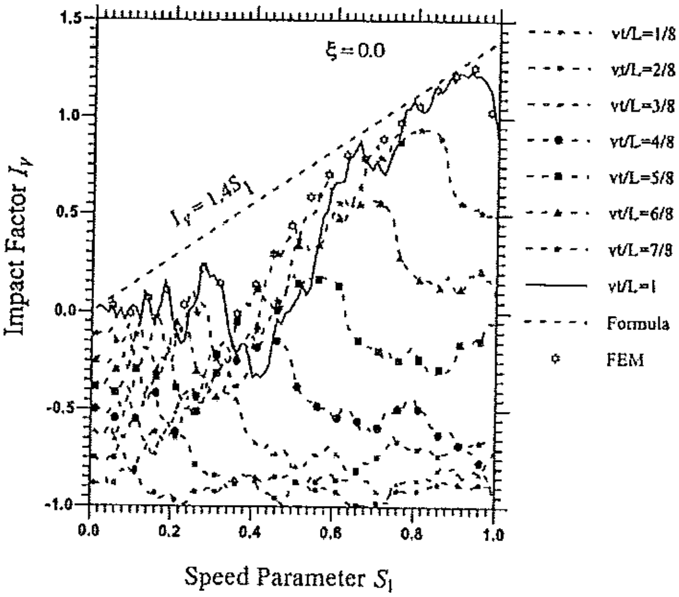


EXHIBIT 5-3
Impact Factors for Shear Force

5.1.1 Transition Spans

TABLE 5-2
ACI Method

Span Length *(ft)*	Speed *(mph)*	Speed *(ft/sec)*	VCF *(Hz)*	f_1 *(Hz)*	Impact *(%)*
197	15	22.00	0.112	1.6	-3.0
197	25	36.67	0.186	1.6	1.6
197	35	51.33	0.261	1.6	6.3
197	45	66.00	0.335	1.6	10.9
197	55	80.67	0.409	1.6	15.6

TABLE 5-3
Yang et. al. Method

Span Length	Speed	Speed	Ω_1	f_1	ω_1	S_1	Impact Deflection	Impact Moment	Impact Shear	Maximum Impact
(ft)	(mph)	(ft/sec)	(Hz)	(Hz)	(rad/sec)		(%)	(%)	(%)	(%)
197	15	22.00	0.351	1.6	10.1	0.035	5.4	4.3	4.9	**5.4**
197	25	36.67	0.585	1.6	10.1	0.058	9.0	7.2	8.1	**9.0**
197	35	51.33	0.819	1.6	10.1	0.081	12.5	10.1	11.4	**12.5**
197	45	66.00	1.053	1.6	10.1	0.105	16.1	13.0	14.7	**16.1**
197	55	80.67	1.286	1.6	10.1	0.128	19.7	15.9	17.9	**19.7**

As seen from Tables 5-2 and 5-3, the calculated impact factors are all less than 30 percent on the Transition spans for Light Rail Vehicle operating speeds of 55 mph. Therefore, the Sound Transit Nominal Impact Factor of 30 percent is valid for the Transition Spans with vehicle operating speeds up to 55 mph. The impact factors increase as light rail vehicle speed increases. Therefore, Light Rail Vehicle operating at speeds higher than 55 mph would require the use of increased impact factors. However, we anticipate that the vehicle speeds will be maintained at or below 55 mph at the Transition Spans.

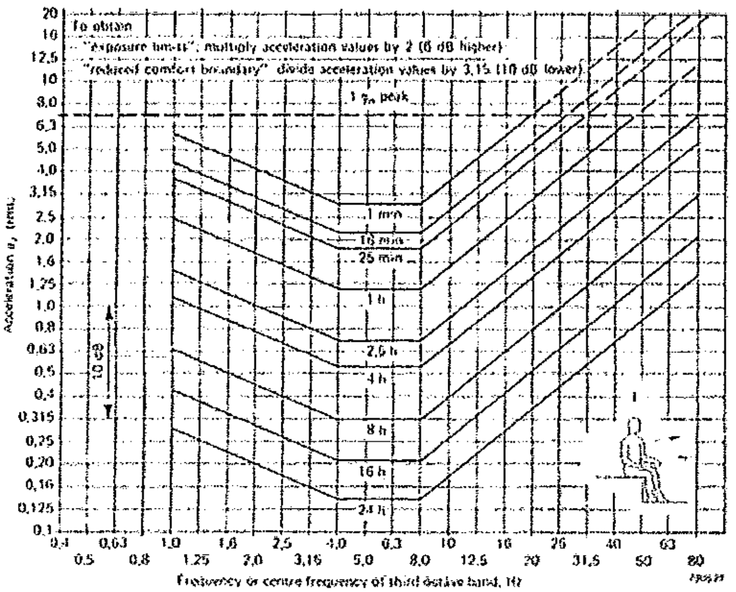
5.2 Deflection Control Analysis Results

Ride quality is usually specified in terms of vehicle accelerations in the passenger compartment. A single specification for the accelerations due only to span dynamic deflections is not usually available for transit systems (the ride quality is usually specified in terms of the overall acceleration levels achieved as a result of all inputs including track irregularities, etc.)..

Sound Transit Design Criteria Manual Section 12.7.6 Ride Quality has the following provisions:

> *Ride Quality: The rms acceleration values shall not exceed the 4-hour, reduced comfort level (vertical) and 2.5-hour reduced comfort level (horizontal) boundaries derived from Figure 2a (vertical) and Figure 3a (horizontal) of ISO 2631 over the range of 1 Hz to 80 Hz, for all load conditions AW0 to AW3.*

From the 4-hour vertical curve as shown in Figure 5-4, the lower bound acceleration can be read as about 0.5 m/sec^2. This value is about 5 percent of g (= 0.5 m/sec^2/9.81m/ sec^2). If the peak accelerations occurring due to the dynamic deflections are less than 0.05g's, they are usually not considered adversely degrading overall ride quality and thus are not significant. In this report, the 0.05g level is used to determine if ride accelerations approach conditions, which are significant.



Vertical vibration exposure criteria curves defining equal fatigue-decreased proficiency boundaries

EXHIBIT 5-4
ISO 2631 Vertical Vibration Exposure Criteria Curves

All of the existing bridges, except the Transition Spans, have load deflections that are less than Sound Transit's requirements of 1/1000. Therefore, only the Transition Spans require a detailed "Deflection Control" analysis. For the live load effects on the structural steel box girders, AASHTO has empirical live load distribution factors. For the deflection calculations of the Transition Spans, instead of using AASHTO distribution factors, a 3-D solid model was created to accurately capture the box girder behavior. Based on two tracks loaded (Figure 5-5) and the span fully-loaded with two car trains, the maximum live load deflection including 30 percent impact is 3.65 inches (Figure 5-6). With a span length of 197 feet, deflection to span ratio is 1/648. This ratio exceeds the than Sound Transit requirement of 1/1000. Therefore, the east Transition Span was further analyzed for dynamic loading caused by a Sound Transit light rail vehicle.

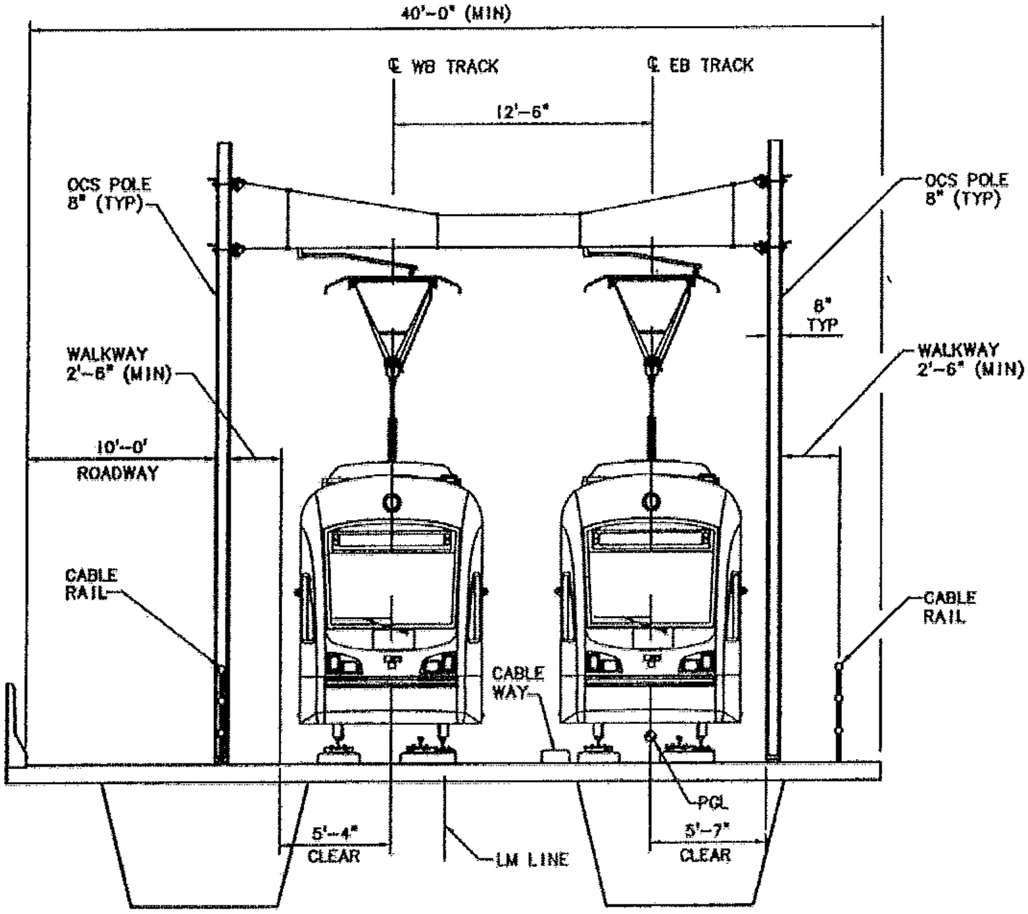


EXHIBIT 5-5
Transition Span Cross Section with Light Rail Vehicle

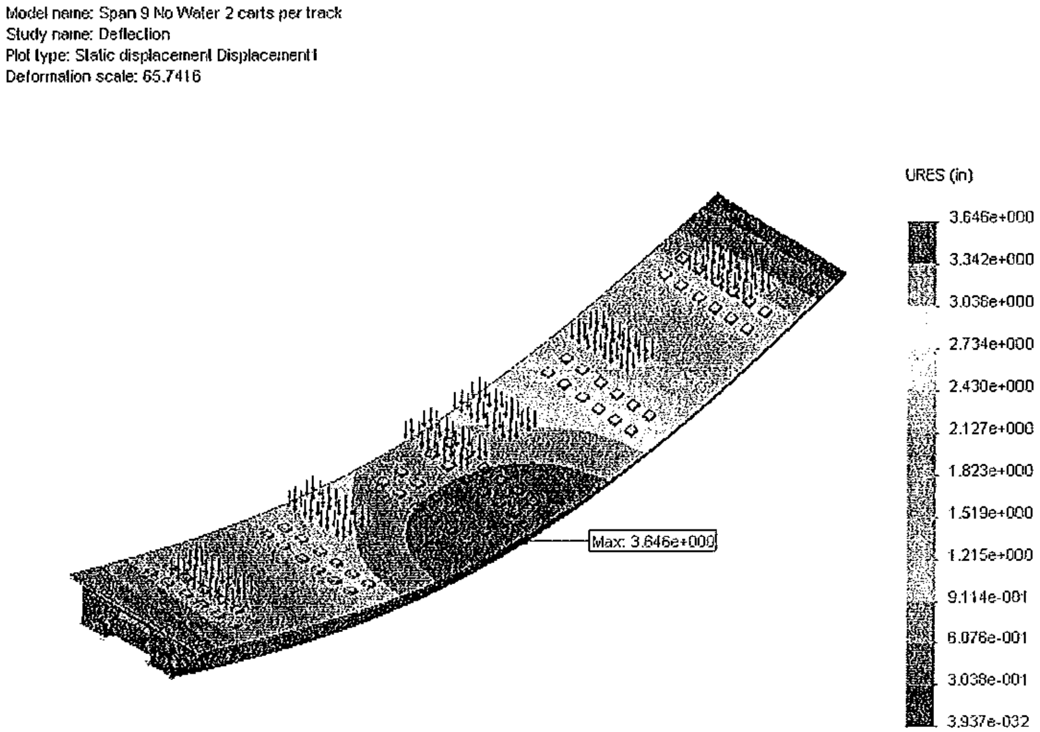


EXHIBIT 5-6
Transition Span Deflection

The dynamic analysis utilized a modal time history method using LUSAS (version 14) finite element software. Span 9, the east Transition Span was modeled. Span 9 consists of two steel box girders, which support an 8.25-inch roadway deck. The model included the weight and mass of the girders, roadway deck, barriers, additional loads due to light rail tracks, pontoon support structure, and 50 percent added hydraulic mass associated with the motion of the pontoon through the water. The added hydraulic mass associated with the motion of the pontoon was based primarily on previous experience at INCA with floating guide wall designs for the U. S. Army Corps of Engineers. The magnitude of added hydraulic mass should be verified when further analysis and design occurs. The fundamental period of vibration of the pontoon and Transition Span in the vertical direction with the added hydraulic mass was found to be approximately 6 seconds. A total of 11 modes were included in the analysis to attain 99 percent mass participation the vertical direction. The finite element model is shown below in (Figure 5-7).

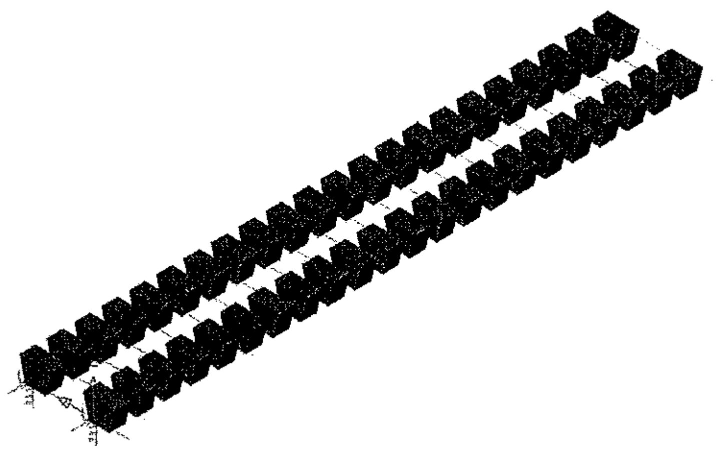


EXHIBIT 5-7
Finite Element Model of Span 9

The model was a centerline frame model of each girder connected intermittently with orthogonal elements to insure participation of both girders when resisting train loads. The train load was run on one track centered on the south girder. The train was run eastward from the floating bridge towards the earth fixed section. Various train velocities were examined ranging from 1 ft/s up to 75 ft/s (50 mph) in increments of 10 ft/s. The results show peak excitations and deflection at midspan of the transition structure on the loaded girder. Peak accelerations ranged from 0.20% g at 5 mph up to approximately 1.8 % g at 50 mph. The range of acceleration results are shown below in (Figure 5-8).

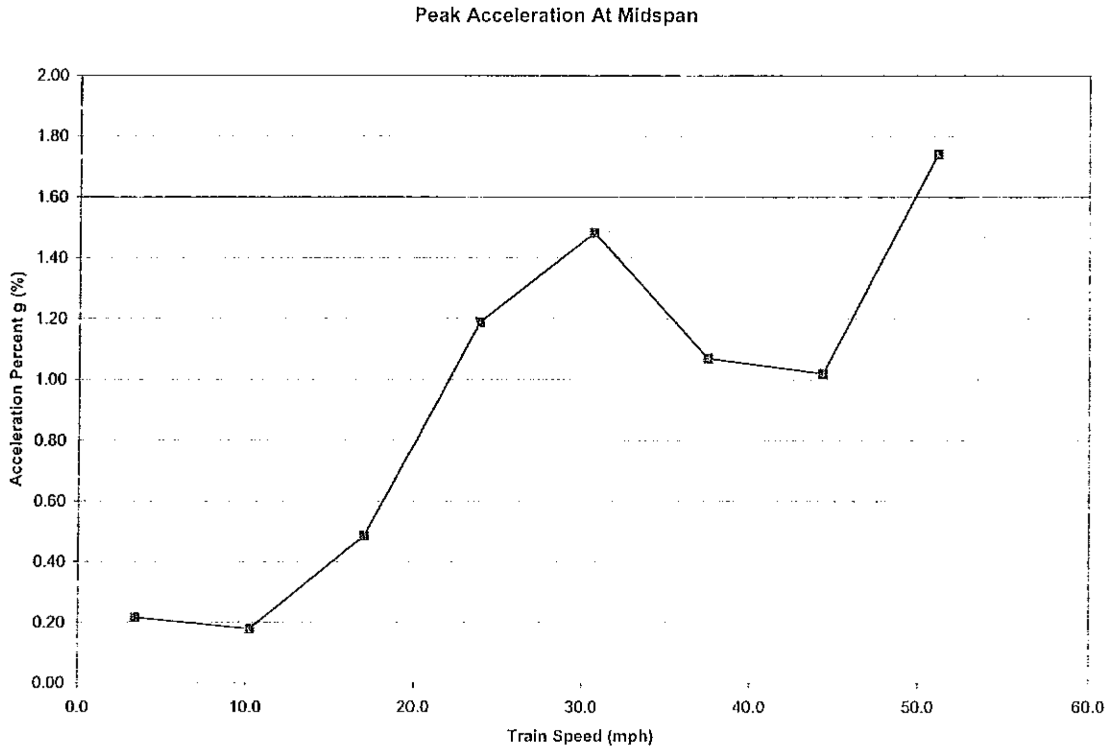


EXHIBIT 5-8
Peak Acceleration at Midspan

A local peak in acceleration occurs around 30 mph at a value of 1.5 % g (Figure 5-9). All results are below the threshold value of 5% of g, which was based on the limit acceleration for rider comfort per ISO 2631 root-mean-square acceleration values for 4-hour reduced comfort level (vertical) boundaries/

The acceleration values are for both the bridge and the light rail vehicle. The mass of the light rail vehicle and suspension characteristics were not considered in the model since the vehicle mass is substantially less than the mass of the bridge and is assumed that its influence on the results will be very minor. The train suspension should serve to further reduce the accelerations experienced by the train passengers.

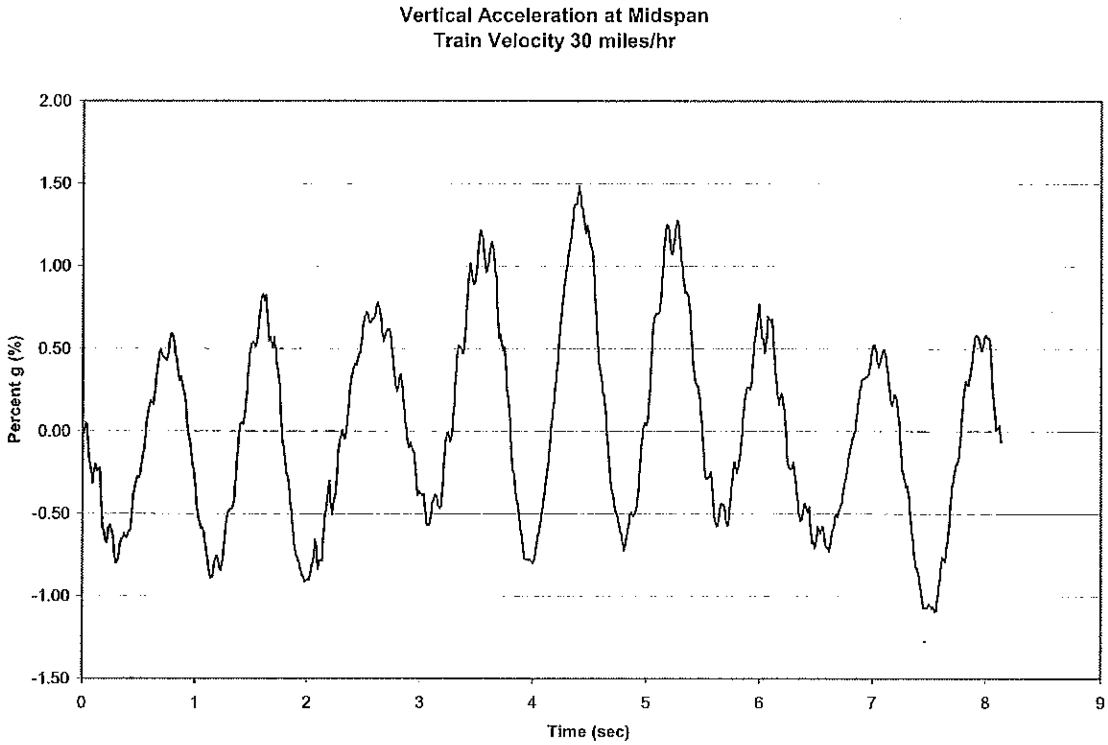


EXHIBIT 5-9
Vertical Acceleration at Midspan

Chapter 6

Conclusion

Structural dynamic analyses were performed on the existing I-90 Bridges between Seattle and Bellevue to verify the WSDOT provided information. Our results confirmed the WSDOT findings on both frequency and deflection checks based on the Sound Transit design manual criteria. While we have not obtained "exactly" the same numbers, the calculated results show similar behavior on all bridges.

A comparison of the WSDOT Study and this study is as follows:

- Transition Spans: Failed to satisfy both "Vibration Control" and "Deflection Control" criteria.
- West Approach Structure: Satisfied "Deflection Control" criteria.
- East Approach Structure: Satisfied "Deflection Control" criteria.
- D2 Viaduct – Concrete Spans: Satisfied "Deflection Control" criteria.
- D2 Viaduct - Steel Spans: Satisfied "Deflection Control" criteria
- Rainier Avenue Overcrossing (Bridge No. 222): Satisfied "Deflection Control" criteria.
- East Channel Bridge: Satisfied "Deflection Control" criteria.

For the Transition Spans, the only bridges that failed to satisfy the Sound Transit's "Vibration Control" criteria, impact factors were calculated to evaluate whether impact loads in excess of the Sound Transit design criteria manual minimum value of 30 percent of live load are required. For vehicle speeds of 55 mph or less, the calculated impact factors are less than 30 percent for the Transition Spans. At operating speeds of 55 mph, the calculated maximum impact factor is 19.7 percent.

An increase of the impact factor would be required for the Transition Spans, if the light rail vehicle operating speed goes over 55 mph. However, we anticipate that the vehicle speeds will be maintained at or below 55 mph at the Transition Spans.

For the Transition Spans, the only bridges that failed to satisfy the Sound Transit's "Deflection Control" criteria, 3-D finite element models were created for further study. These models were used to study the acceleration levels due to light rail passage in order to evaluate rider comfort and determine if the accelerations approach levels that significantly degrade the ride quality. We found that at a 55 mph operating speed the accelerations reached about 2.0% g level, which is less than the threshold value of 5.0% g limit acceleration for rider comfort per ISO 2631.

As the Eastside HCT Project advances into the preliminary and final design phases, the consultant team will be re-evaluating the vehicle-bridge interaction in more detail. If required, more refined analysis models will be created to investigate the Vehicle-Structure Interaction Dynamic Analysis in order to insure the structural integrity and comfort of the riders. Mitigation measures will be implemented if the analysis determines that it is necessary.

EAST LINK PROJECT

Stray Current Analysis Report

December 2007

SOUNDTRANSIT

CENTRAL PUGET SOUND REGIONAL TRANSIT AUTHORITY

SOUND TRANSIT

SOUND TRANSIT EAST LINK PROJECT
Phase 2

Stray Current Analysis Report

Prepared for:
CH2M HILL, Inc. and Sound Transit

Prepared by:
PARSONS

December 2007

Translation services and information in accessible formats are available upon request by calling 1.800.201.4900 (voice) or 206.398.5410 (TTY).

For more information about the Stray Current Analysis Report call (refer to specific community relations coordinator as appropriate) or write Sound Transit, 401 South Jackson Street, Seattle, WA 98104-2826. You may also e-mail Sound Transit at main@soundtransit.org, visit our Web site at www.soundtransit.org or call our toll free information line at 1-800-201-4900.

Contents

Chapter 1

Introduction

The Sound Transit East Link Project will be approximately 18 miles of double track over five Segments. The 90% Conceptual Engineering Plans include various alternatives within the segments addressing different options for elevated, at-grade, and below grade track construction. Design considerations are also included for four maintenance facility sites and six new or enhanced park-and-ride facilities.

1.1 Analysis Objectives

The purpose of this Analysis is to define requirements for corrosion control designs for Phase 2 of the Sound Transit East Link Project. The following proposed design elements were evaluated:

- Traction Power Design
- Existing reinforced concrete structures to be retrofitted with LRT tracks
- New reinforced concrete LRT structures
- Trackwork configurations
- Utility piping systems in proximity to the proposed alternative alignments

As-built structural drawings for the Lake Washington Floating Bridge were reviewed in conjunction with Segment A 90% Conceptual Plans for LRT track placement on existing reinforced concrete structures and within existing tunnels. The Conclusions and Recommendations herein are based on existing Sound Transit corrosion control practices, and industry-accepted methods for controlling stray current and soil corrosion on buried and embedded metallic structures associated with LRT operations.

1.2 Soil and Stray Current Corrosion

Corrosion of metals is often described a "the destruction of a metal, or its properties because of a reaction with its surroundings (environment)." Although there are numerous environments that are corrosive to metals, the scope of this Analysis is corrosion of metals in soils and concrete.

1.2.1 Soil Corrosion

Soil Corrosion for this Analysis is defined as "the deterioration of a buried metallic structure due to the natural corrosivity of the soil in contact with its external surfaces." Some corrosive properties of soils can include:

- Low electrical resistivity (high ionic content)
- Low pH (acidic)
- Differences in aeration or soil types between two locations on a metallic surface

1.2.2 Stray Current Corrosion

The definition of Stray Current Corrosion for this Analysis is, "Corrosion of a buried or embedded metallic structure caused by differences in electrolytic voltage potentials between two locations on the structure surface due to the presence of stray dc current." The source of stray current from LRT operations is the dc negative return circuit, most of which is the running rails.

1.3 Structures Affected by Soil and Stray Current Corrosion

All metallic structures buried in soil or embedded in concrete will eventually corrode. Corrosion and stray current control measures should be incorporated into all systems and structures associated with LRT systems to protect Sound Transit and WSDOT-owned structures and public utility piping systems from premature corrosion failure.

1.3.1 Reinforced Concrete Structures

Stray current control designs for steel-reinforced concrete structures focus on protection of trackway structures since the running rails they support are the source of stray current. These structures include:

- New and/or existing bridge structures with embedded track
- New and/or existing bridge structures with direct fixation track
- New and/or existing tunnels with direct fixation and/or embedded track
- New at-grade embedded track slabs

Reinforced concrete structures adjacent to trackway structures such as cast-in-place retaining walls and roadway bridges do not require stray current control designs.

1.3.2 Buried Utility Piping Systems

Buried utility piping systems are subject to both soil and stray current corrosion. Stray current and corrosion control designs focus on pressurized metallic pipelines such as water and natural gas piping. Corrosion of non-pressurized piping systems is not as critical as for pressurized systems since they can sustain much greater corrosion deterioration without failure.

1.3.3 I-90 Floating Bridge

The fact that the I-90 Bridge is a floating structure does not increase the risk of stray current corrosion damage. However, alternative stray current monitoring capability will be required to ensure the long-term effectiveness of the insulating track fasteners.

Chapter 2

Corrosion Control Methods

2.1 Source Control

Source control is the most effective means to mitigate stray current activity on buried and embedded metallic structures in proximity to an LRT system. The magnitude of stray current a system generates is directly proportional to the voltage potential between the negative return circuit and ground, and inversely proportional to the resistance between the negative circuit and ground. Therefore, the intent of source control measures is maintain low voltage potentials in negative return circuit (running rails) and to maintain high electrical resistance between the running rails and ground.

2.1.1 Trackwork

Electrical isolation of the running rails from ground is critical to the effectiveness of the source control system. Modern track-fastening system designs are extremely effective at reducing the generation of stray current from LRT operations. Insulating track fasteners for concrete tie-and-ballast track construction and direct fixation (DF) fasteners on reinforced concrete track slabs are designed to provide between one and ten megohms resistance. Insulating materials for embedded track systems provide a minimum volume resistivity of 10^{12} ohm-centimeters. These high levels of insulation provide very high track-to-earth resistance values.

- Tie-and-ballasted track with high resistance track fasteners on concrete ties provides track-to-earth resistances greater than 500 ohms per 1000 track feet.
- Direct fixation track with high resistance rubberized track fasteners provides track-to-earth resistances greater than 1,000 ohms per 1000 track feet.
- Embedded track using various methods of rail encapsulation such as rail coatings, polyurethane encasement, and rail boots provides track-to-earth resistances greater than 200 ohms per 1,000 track feet.

The resistances are for clean, dry fastening systems. Moisture reduces the level of insulation. However, if kept clean, these insulating track fastening systems should maintain levels greater than 100 ohms per 1000 track feet. By comparison, non-insulated timber tie-and-ballast track starts around 40 ohms per 1000 track feet when new, but shortly are only capable of resistance values in the single digits, and are effectively grounded when wet.

2.1.2 Traction Power Design

Voltage potentials between the negative circuit and ground are reduced through the use of continuously welded rail, periodic cross bonding between rails, and closely spaced substations. Substation placement with respect to areas where train acceleration is likely to occur is another factor considered during traction power design to lower voltage rise in the running rails. Optimum substation spacing defined by traction power simulations normally provides acceptable rail voltage potentials for stray current control.

2.2 Corrosion Monitoring and Mitigation Measures

Corrosion monitoring and mitigation measures differ from source control in that they deal with corrosive conditions of concrete or soils rather than actively preventing corrosive conditions to occur.

Although source control techniques are used on the LRT systems, some stray current is likely to enter the concrete and/or earth, although at greatly reduced levels. The corrosivity of soils around buried pressurized piping systems is also considered in addition to potential stray current activity.

2.2.1 Reinforced Concrete Trackway Structures

Standard practice for monitoring stray current activity on new reinforced concrete trackway structures is to selectively weld the top layer reinforcement in the trackslab, bridge deck, or tunnel invert. The intent of this practice is to reduce the flow of electrolytic current between discontinuous reinforcing bars within the trackway. Grounding systems are also provided to allow an electrical path for stray current to pass from the reinforcement to earth. *However, the main purpose for this strategy is to provide a means to monitor stray current activity on the structure. Effective insulating track fastening systems are the key to controlling stray current corrosion on reinforced concrete trackway structures.*

Test stations associated with the grounding systems are used to monitor stray current activity on the bonded reinforcement. This provides a means to evaluate the effectiveness of the track fastening system. Existing bridges or tunnels retrofitted with LRT have no means for bonding the reinforcement. The effectiveness of the track fasteners on existing structures can be evaluated using test facilities on adjacent new structures and by performing periodic track-to-earth resistance tests. In the case of the I-90 Floating Bridge where the only adjacent structures are at either shore, methods to remotely monitor track-to-earth resistances across the bridge will be implemented. This will give a direct indication of the condition of the track fasteners without monitoring stray current activity on embedded reinforcement.

2.2.2 Buried Utility Piping

Cathodic protection systems are required for new, metallic, pressurized piping systems where failure caused by corrosion may affect public safety and/or LRT operations. This includes new gas and waterlines routed below LRT tracks, and/or piping systems whose proximity to the tracks would not allow for repair work without impacting LRT operations. Cathodic protection system components include:

- Galvanic anodes
- Electrical isolation at connections to existing piping
- Pipe coatings
- Bonded mechanical pipe joints
- Permanent test facilities

The test facilities provided to evaluate the cathodic protection system components can also be used to monitor stray current activity on the piping system.

2.3 Sound Transit Corrosion Control Methods

2.3.1 Design Criteria

Review of the Sound Transit Corrosion control Design Criteria, 2005 Edition, Revision 0 indicates compliance with standard corrosion control practices including:

- Stray current control systems for reinforced concrete LRT structures
- Cathodic protection for buried metallic piping systems
- Electrical isolation of trackwork from ground

- Electrical isolation of the traction power dc circuits from ground

2.3.2 Standard Drawings

Review of the Link Light Rail Project standard corrosion control drawings dated 11/21/01 and 7/12/02 show specific details of the corrosion control requirements outlined in the Design Criteria:

- Cathodic protection details for buried metallic piping systems
- Bonding details for reinforced concrete LRT structures
- Track isolation monitoring facilities

Chapter 3

East Link System Description

The Sound Transit East Link Project will utilize numerous different types of trackway structures and track fixation techniques. The information contained in this Chapter is based on review of the 90% Conceptual Engineering Plans.

3.1 Segment A

The LRT trackway will be retrofitted into existing roadways and reinforced concrete structures in Segment A. These structures include:

- Embedded track retrofitted on existing aerial structures and tunnels
- Embedded track added to existing HOV roadways
- Direct fixation track retrofitted on existing aerial structures
- Direct fixation track retrofitted into Mount Baker and Mercer Island Tunnels
- Direct fixation track retrofitted on the I-90 Homer Hadley Floating Bridge

Numerous major waterlines cross the proposed alignment.

3.2 Segment B

There are five alternative alignments under consideration for Segment B. Proposed reinforced concrete trackway structures include:

- Direct fixation track retrofitted on the I-90 East Channel Bridge
- At-grade embedded trackslabs
- New direct fixation aerial structures

Alternative B7 includes extensive at-grade tie-and-ballasted track and approximately three major waterlines crossing the alignment.

3.3 Segment C

There are six alternative alignments under consideration for Segment C. Proposed reinforced concrete trackway structures include:

- At-grade embedded trackslabs
- Direct fixation track in retained cut structures
- Direct fixation track in cut-and-cover tunnels
- Direct fixation track on retained fill structures
- Direct fixation in bored tunnels
- Direct fixation on aerial structures

The number of major waterlines crossings is dependent on which alignment is used.

3.4 Segment D

There are four alternative alignments under consideration for Segment D. Proposed reinforced concrete trackway structures include:

- At-grade embedded trackslabs
- Direct fixation track on aerial structures
- Direct fixation track in concrete trench structures

Tie-and-ballasted track on retained fill is an option in place of aerial structures. Major gas, petroleum, and water pipelines cross the different alternative alignments.

3.5 Segment E

There are three alternatives under consideration for Segment E. Proposed reinforced concrete trackway structures include:

- Direct fixation track on retained fill structures
- Direct fixation track in concrete trench structures
- Direct fixation track in cut-and-cover tunnels
- Direct fixation track on aerial structures
- At-grade embedded trackslabs
- At-grade direct fixation track systems

There are no major waterlines shown crossing the alignment.

3.6 Traction Power System

Sound Transit uses a 1500 Vdc instead of the more common system voltage of 750 Vdc for light rail systems. Initial traction power simulations have been performed for the East Link Project using nominal 1500 Vdc traction power substations. The increased voltage allows for longer distances between substations without increasing rail-to-earth voltages. Substation spacing will generally range between 1.5 and 3 miles apart. Other considerations used to reduce voltage rise in the rails include:

- Placing the substations adjacent to passenger stations where practicable to account for increased traction current demand
- Placing substations in proximity to each shoreline for the I-90 floating bridge to reduce voltage rise of the rails on the bridge structure

In addition to these design considerations, most of the proposed trackway will be on dedicated or semi-dedicated aerial structures or in tunnels, which reduces the incidence of voltage rise from repeated starting and stopping at grade crossings. All design considerations should be addressed during subsequent design phases to optimize traction power performance and stray current control.

Chapter 4

Conclusions

4.1 Sound Transit Corrosion Control Policy

Based on review of Chapter 17, Corrosion Control, of the 2005 Sound Transit Design Criteria and the Corrosion Control Standard Drawings for the Link Light Rail Project, Sound Transit is committed to the importance of stray current and corrosion control designs for Sound Transit LRT Systems.

4.2 New Reinforced Trackway Structures

Current state-of-the-art stray current control systems, including available effective track fastening systems, can be incorporated into all proposed new trackway structures.

4.3 LRT Tracks on Existing Structures

Adequate stray current control can be achieved on existing reinforced concrete structures retrofitted with LRT tracks through the use of insulating track fastening systems and Track Isolation Monitoring Facilities developed for the Link Light Rail Project.

Chapter 5

Recommendations

5.1 Traction Power System

The following methods should be used to reduce the magnitude of stray current generated by LRT operations:

- The traction power system should be operated with no direct or indirect electrical connections between the positive and negative traction power distribution circuits and ground, with the exception of temporary ground clamping if required to mitigate excessive voltage rise in the rails.
- Place traction power substations adjacent to passenger stations if practicable.
- Place traction power substations as close as practicable to each shoreline of the I-90 floating bridge.
- Coordinate traction power designs with stray current control requirements.

5.2 Electrical Isolation of Trackwork

The following methods should be used to electrically insulate the running rails from ground:

- All embedded trackwork including grade crossings should use a rail boot system to isolate the entire rail surface from contact with embedment material or grade crossing panels.
- All direct fixation track installed in tunnels, aerial sections, trenches, retained cut, and retained fill structures shall use insulating track fasteners to isolate the rails from the track inverts.
- All mainline tie-and-ballast trackwork should use insulating track fasteners on concrete ties.
- All special trackwork installed on timber ties should use insulating track fasteners.

5.3 Bonding of Reinforced Trackway Structures

The following methods should be used for bonding new reinforced trackway structures:

- Aerial structures should have the top layer reinforcement welded for electrical continuity. Install grounding provisions and test facilities at each end of the structure and at intermediate intervals to facilitate stray current testing.
- Tunnel, trench, retained cut, and retained fill structures should have the top layer reinforcement welded for electrical continuity. Install test facilities at each end of the structure and at intermediate intervals to facilitate stray current testing.
- Embedded trackslabs should have the top layer reinforcement below the rails welded for electrical continuity. Install test facilities at each end of the structure and at intermediate intervals to facilitate stray current testing.

5.4 Buried Utility Piping Systems

The following corrosion control methods should be used on buried piping systems in proximity to the LRT system:

- Sound Transit-owned water service piping and firelines for maintenance facilities or LRT Stations should be cathodically protected.
- All buried, metallic, pressurized piping systems regardless of the owner/operator should be cathodically protected if failure of the piping would affect public safety or continuity of LRT operations.

5.5 Existing Structures Retrofitted with LRT Tracks

Since bonding reinforcement is not possible on existing reinforced concrete structures, the following special considerations should be implemented for the existing aerial structures, tunnels, and the I-90 floating bridge in Segment A:

- In addition to using standard insulating direct fixation track fastening systems, additional isolating provisions should be considered such as epoxy coating the top of the rail plinths or using oversized insulating pads under the track fasteners.
- Install remote track isolation monitoring facilities along the trackway in Segment A to monitor track-to-earth resistance on a regular basis.
- Implement an aggressive track inspection and cleaning program for Section A, using data collected by the remote track isolation monitoring facilities to pinpoint areas requiring special attention.

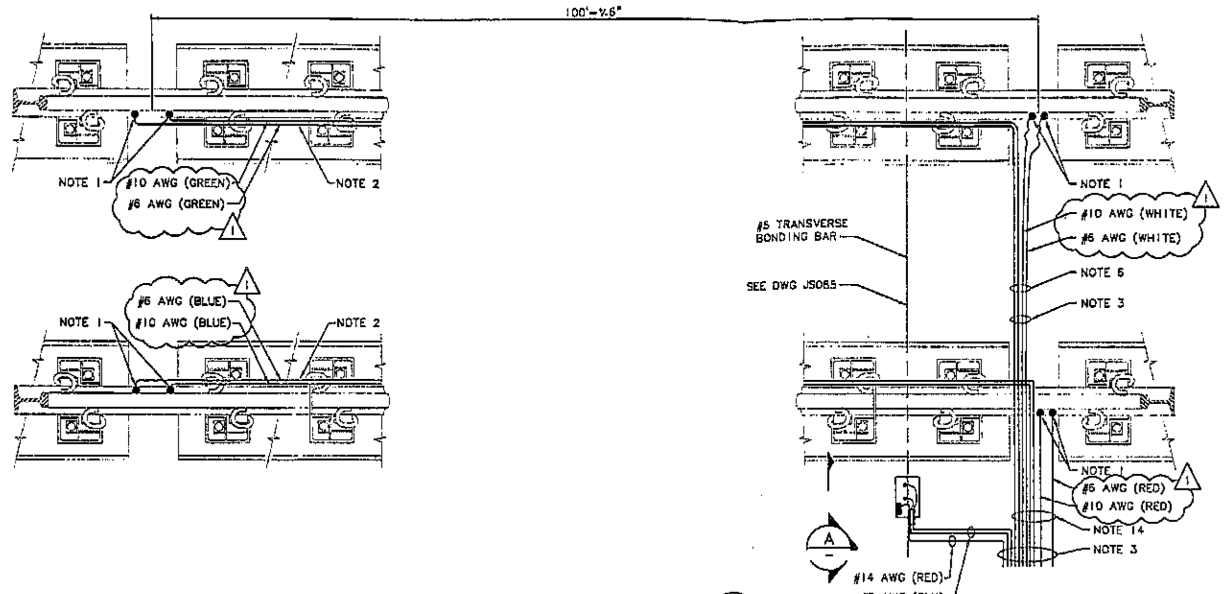
5.5.1 I-90 Floating Bridge

The I-90 Floating Bridge in Segment A is unique compared to other existing reinforced concrete structures that will be retrofitted with LRT tracks. Access to the bridge for performing track-to-earth testing is limited and there are no adjacent structures beyond the shorelines to assist in monitoring stray current activity. The stray current control system for the bridge will include a track fastening system with a high level of electrical isolation, a remote track isolation monitoring system, and an active track monitoring and maintenance program carried out by Sound Transit maintenance personnel.

- The track isolation system will include the use of insulating direct fixation (DF) track fasteners. During the design phase, the fasteners will be tested under various moisture conditions to determine if additional measures are required to enhance the performance of the fastening system. Additional measures may include an extra insulating "sock" installed between the rail base and DF fasteners, an additional insulating pad placed between the DF fastener and rail plinth, dielectric coatings on the plinths and/or deck, and possibly using non-metallic reinforcement in the rail plinths. The final configuration of the insulating track fastening system will be determined based on field-testing of a track model under controlled simulations of various moisture conditions.
- The remote track isolation monitoring system will be based on the Sound Transit design for the Link Light Rail Project. Sound transit plans to implement this system on existing track sections prior to the design phase for the East Link Project. A copy of the Corrosion Control Standard Drawing - Aerial Guideway Corrosion Control Test Box Wiring Layout from the Link Light Rail Project is included in the Appendix of this Report. The actual configuration and number of test points will be determined during preliminary design stages.
- Sound Transit maintenance personnel will be trained to operate the remote track isolation monitoring system and to evaluate the data recorded by the test equipment located in a TPSS adjacent to the bridge. Periodic visual track inspections will be performed in conjunction with the remote monitoring system to identify contaminated or defective insulating components, which will be cleaned, repaired, or replaced by Sound Transit.

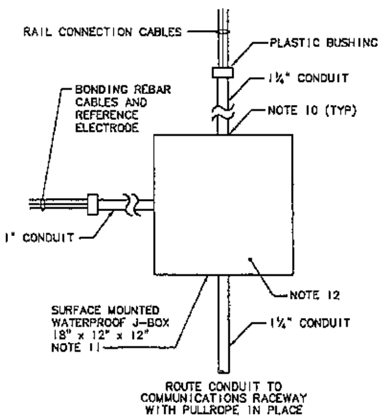
Appendix

Aerial Guideway – Corrosion Test Box Wiring Layout



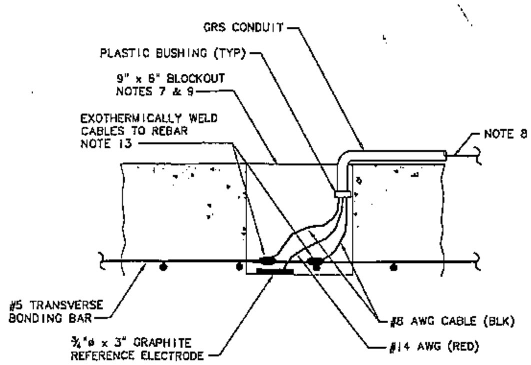
TEST CABLE CONFIGURATION (1 / JS082)

SCALE: NTS



CORROSION CONTROL JUNCTION BOX (2 / JS082)

SCALE: NTS



SECTION (A / -)

SCALE: NTS

NOTES:

1. EXOTHERMICALLY WELD CABLES TO NEUTRAL AXIS OF RAIL AND COAT ALL EXPOSED COPPER WITH COAL TAR EPOXY.
2. PROVIDE RAIL CABLE RETAINER CLIPS AT 18" MINIMUM SPACING. ROUTE CABLE TO AVOID CONFLICT WITH PANDROL CLIPS.
3. BUNDLE CABLES AT 30" MINIMUM SPACING WITH APPROVED HEAT SHRINK SLEEVES AND ROUTE TO CORROSION CONTROL TEST BOX.
4. ALL CABLES TO HAVE XHHW INSULATION.
5. PROVIDE TEST WIRE CONNECTIONS FOR NORTHBOUND AND SOUTHBOUND TRACKS. TAG CABLES AND JUNCTION BOX "NB" OR "SB".
6. PLACE CABLES BETWEEN RAILS IN 1" GRS CONDUIT. SURFACE MOUNT CONDUIT WITH PLASTIC BUSHING AT EACH END.
7. PATCH BLOCKOUT WITH APPROVED CEMENTICIOUS GROUT.
8. ROUTE CABLES TO CORROSION CONTROL TEST BOX.
9. DEVELOP DIMENSIONS FOR BLOCKOUT BASED ON GIRDER REINFORCING CONFIGURATION.
10. SEAL CONDUIT WITH APPROVED SINGLE COMPONENT SILICONE ELASTOMER.
11. FASTEN JUNCTION BOX TO GIRDER BETWEEN NORTHBOUND AND SOUTHBOUND TRACKS AS DIRECTED BY THE RESIDENT ENGINEER.
12. IDENTIFY AND TAG ALL CABLES. PROVIDE A MINIMUM OF 48" OF EXCESS CABLE NEATLY COILED AND BUNDLED.
13. PRE-FABRICATED CONNECTIONS MAY BE SUBSTITUTED FOR EXOTHERMIC WELD. SEE DWG JS088 (REPLACE #2 AWG SHOWN WITH #8 AWG).
14. PLACE CABLES IN 1¼" GRS CONDUIT WITH PLASTIC BUSHING AT RAIL. SURFACE MOUNT AND ROUTE TO CORROSION CONTROL JUNCTION BOX.

No.	Date	Dsn.	Chk.	App.	Revisions
1	07/12/02	NDB	DDR	DLP	CHANGED WIRE COLORS

Designed By: D. BURKE
Drawn By: M. MOSS
Checked By: D. ROSS
Approved By: D. PORTER

LTK Engineering Services

Submitted: D. PORTER
Date: 07/12/02
Approved:
Date:

SOUNDTRANSIT

Scale: NTS
Filename: Js082.dwg
Contract No.: ST STS
Date: 11/21/01

LINK LIGHT RAIL PROJECT
CORROSION CONTROL
STANDARD
AERIAL GUIDEWAY
CORROSION TEST BOX WIRING LAYOUT

Drawing No.: JS082
Sheet No.: 20
Rev.: 1

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EAST LINK PROJECT

Overhead Catenary System Concept Study Final Draft Report

December 2007

SoundTransit

CENTRAL PUGET SOUND REGIONAL TRANSIT AUTHORITY

SOUND TRANSIT

SOUND TRANSIT EAST LINK PROJECT
Phase 2

Overhead Catenary System Concept Study

Final Draft Report

Prepared for:
CH2M HILL, Inc. and
Sound Transit

Prepared by:
Parsons

December 2007

Translation services and information in accessible formats are available upon request by calling 1.800.201.4900 (voice) or 206.398.5410 (TTY).

For more information about the Overhead Catenary System Concept Study Draft Report call (refer to specific community relations coordinator as appropriate) or write Sound Transit, 401 South Jackson Street, Seattle, WA 98104-2826. You may also e-mail Sound Transit at main@soundtransit.org, visit our Web site at www.soundtransit.org or call our toll free information line at 1-800-201-4900.

Executive Summary

The proposed East Link Light Rail Transit (LRT) network being considered by Sound Transit is a conventional light rail system that will interconnect with Sound Transit's Central Link corridor at the International District in Seattle. It will then transition to an I-90 alignment, across Lake Washington and through Mercer Island, and then on through the cities of Bellevue and Redmond. Past the I-90 portion of the alignment (Segment A), several alternative alignments are still under consideration. This report will address the OCS system to be used in general, with special emphasis on the I-90 floating bridge, and the requirement for installation in the I-90 and downtown Bellevue tunnels.

A 1,500 Volt Direct Current (1,500 vdc) traction power system has been selected by the Traction Power Task of the Alternatives Analysis. The reasons for the selection include its compatibility with Sound Transit's existing LRT system (particularly important at the central Link interconnect) and compatibility with the existing LRT vehicles. The long stretches to be fed across Lake Washington and Mercer Island also require the use of the 1,500 vdc system in order to maintain sufficient voltage for LRT operations, particularly under contingency conditions.

For similar reasons, the study of the candidate OCS system focused on maintaining the existing styles of OCS equipment in order to maintain compatibility with the existing system. East Link will extend approximately 18 miles from the ITS Central Link LRT interconnect in Seattle. It would be problematic for future maintenance to deviate from the existing types of OCS already in use, provided they could accommodate the new service. This would avoid stocking additional inventories of spare parts, providing response and repair fleets with a second inventory of equipment, and avoid retraining maintenance crews on the new styles of OCS. It also provides an additional benefit in that the existing conductor sizes could be used to optimize substation spacing within the existing criteria, and would be able to accommodate the long reach across Lake Washington. It is also noted that the 500/350 OCS system has become the defacto standard in North America, and is used on most of the major LRT systems. This provides a well-experienced supplier and installer base, as well as competitive pricing for procurement and installation.

Therefore, the existing conductors of a 500 kcmil copper messenger supporting a 350 kcmil grooved contact wire are recommended for reuse on the East link Corridor.

1.0 General Discussion

The purpose of an Overhead Contact System (OCS) is to distribute Traction Power from the wayside Traction Power Substations to the rolling stock as they traverse the alignment. Therefore, the OCS system must be able to provide satisfactory electrical performance over the range of climatic conditions and alignment characteristics experienced over the length of the system extension. The relevant criteria are spelled out in Sound Transit's Design Criteria Manual for the North Link (2005 edition), which has been used as the basis for this study.

The climatic criteria specified are common in the industry, and no exception is foreseen at this time. Similarly, the electrical characteristics are based on the operational requirements of the system, and no deviations are foreseen.

As the same rolling stock will be used, deviations to clearance requirements are not anticipated. Current design calls for direct fixation rail in the existing tunnels. Embedded tracks are also under consideration. A contact wire height of between 15 and 16 feet can readily be obtained which will allow for normal over-the-road vehicular maintenance equipment or rail mounted equipment.

2.0 Selection of Conductors

Current Sound Transit (ST) Standards and Design Criteria indicate the use of a 500 kcmil Copper Messenger and a 350 kcmil grooved copper trolley wire for use as the conductors comprising the OCS. This conductor combination provides a RMS ampacity of 1295 Amperes, with a 30% worn trolley wire, which has been verified as acceptable by the Traction Power Simulation Report.

This combination of conductors has successfully been used by ST before, and the maintenance department is now familiar with this type of OCS, and maintains an inventory of spare parts for it. Reuse of these conductors would simplify the spares inventory requirement, and avoid confusion during emergency restorations or routine wire replacement in the future.

The technical sheets for current Sound Transit construction packages indicate that this conductor combination provides a judicious balance of weights, tensions, strengths, and span lengths to meet the design and operational criteria. There is no need for more expensive materials to increase strength requirements, particularly since they tend to diminish electrical capacity. Similarly, there is no need to increase conductor sizes since the ampacity is adequate for the electrical service requirements.

The 500/350 combination has become a defacto standard in the North American LRT industry, and has been used on many major projects. This provides an experienced supplier base, which allows for good competition in component procurement. It also provides service-proven hardware, and the opportunity to solicit competitive bids from several installers who are used to working with the equipment, and know what it takes to provide a successful and economical installation. The use of service-proven hardware also provides Sound Transit with the advantage of not having to serve as a test bed for suppliers to use for product development, and provides materials and assemblies that have had the "kinks worked out."

3.0 Alignment Discussion

There are several types of alignment that must be considered in the selection of appropriate styles of OCS. For the ELT they include open route, joint use with busway, street running, tunnel, floating bridge, and maintenance facility. Each is discussed in more detail below.

3.1 Open Route

Sound Transit's Design Criteria calls for Automatically Tensioned Simple Catenary (ATSC) in the open route areas. Open Route, for the purpose of this study, is considered to be where the LRT is on its own alignment, or track structure, and not shared with vehicular traffic or in a tunnel. The use of ATSC is common in this type of application and lends itself to optimization of span lengths (i.e. distance between supporting structures) and conductor sizes and strengths. The sizes of the support structures can also be optimized to reduce the number of types and sizes of poles, thereby controlling installation and inventory costs.

The catenaries on the poles are supported by hinged cantilevers (see figure 3-1), which can rotate along track to accommodate conductor expansion and contraction due to temperature and ice and wind loading changes. Temperature changes are governed by both climatic conditions (ambient temperature and solar heating) and electrical heating due to traction current flow.

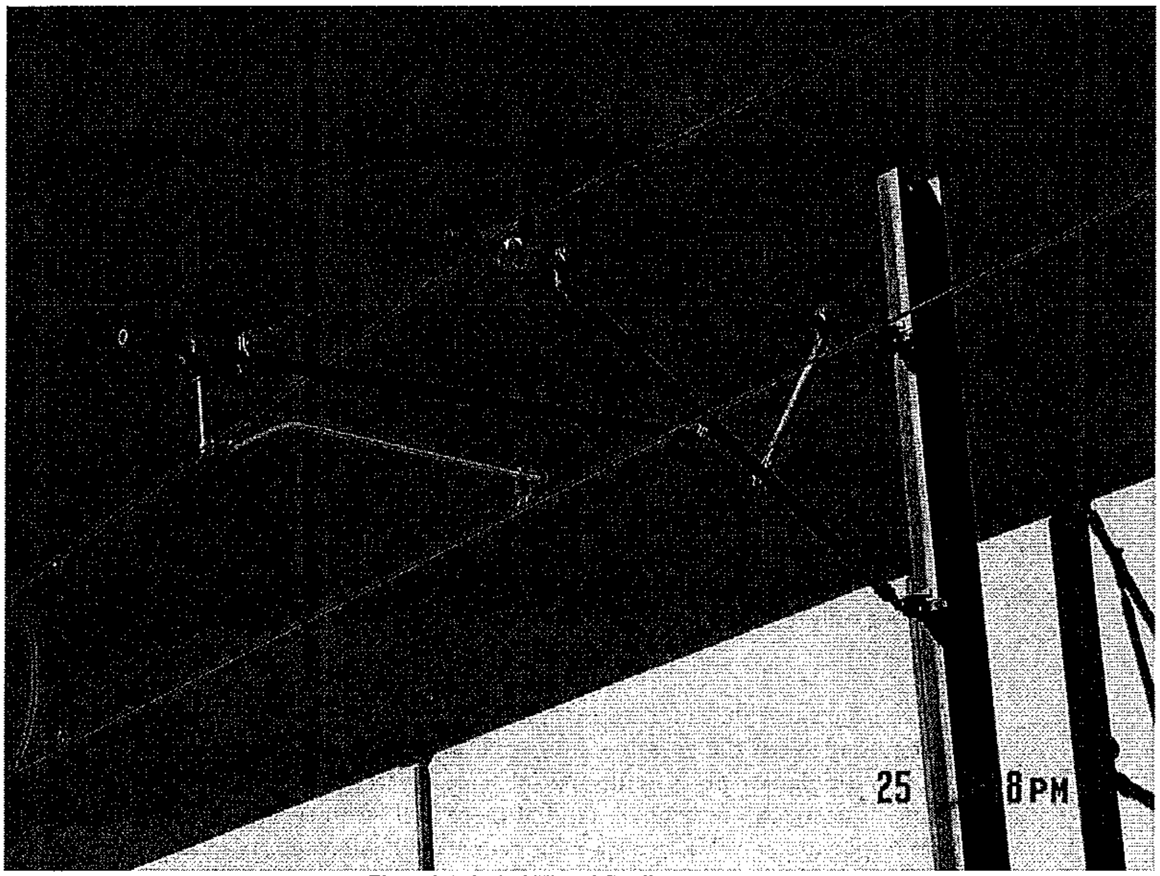


Figure 3-1 Typical Hinged Cantilever

Within the majority of the temperature and ice loading range the tension in the conductors is held constant by the use of sets of weights and pulleys (see Figure 3-2). As the conductors expand and contract, the weights move up and down the poles and the hinged cantilevers rotate to accommodate the movement. The distance between the weight sets is determined by the amount of rotation on the cantilevers that can be accommodated and maintain pantograph security. Pantograph security is a complex issue that can be simply defined as keeping the contact wire within an allowable area of the pantograph head under the most adverse climatic conditions.

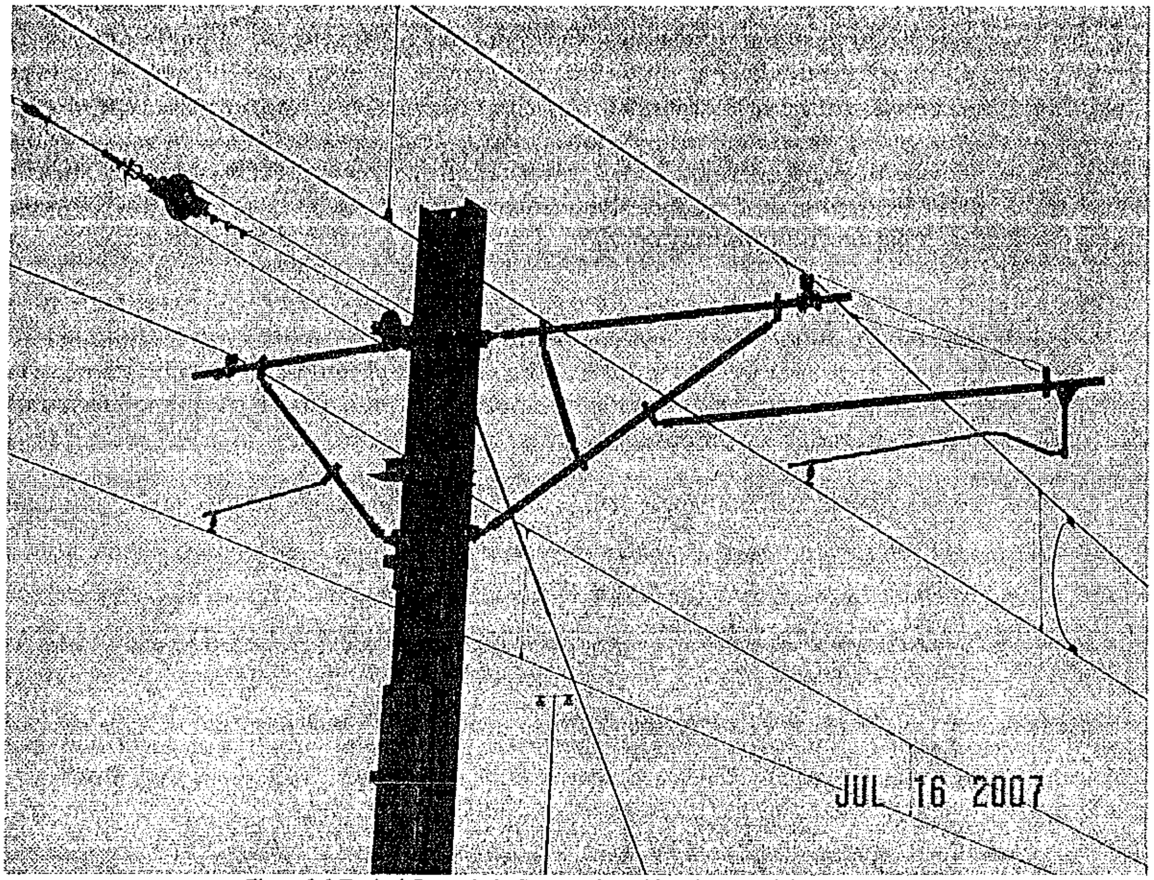


Figure 3-2 Typical Center Pole Construction with Balance Weight

The advantage of maintaining a constant tension in the wires is reduced loading on the poles, improved dynamic performance of the catenaries and pantographs (important with 4 car consists), and more efficient use of the strength of the conductors. An additional benefit is that the wires do not sag as much under higher temperatures or ice loading, which allows the use of shorter structures to maintain minimum wire heights. This not only reduces costs and structure spacing, but also reduces the visual impact of the OCS.

In accordance with the Design Criteria, standard galvanized wide flange poles are recommended for the system. In special areas, the use of other types of poles (such as tapered tubular or poles with special features to blend with WSDOT's "Mountains to Sound" vision) can be considered, but these will be an exception to the norm due to their cost implications. The poles will generally be located between the tracks, thereby reducing pole quantities, and the visual "tunnel effect" caused by locating catenary poles opposite each other on the outside of the tracks.

3.2 Joint Use with Busway

Trains traversing the alignment out of the International District will be drawing significant traction currents due to the sustained gradient through the ramp area. The sharing of the ROW with busses will also require the OCS to be kept at a roadway height over the tracks and busway. For this reason, it is recommended that an ATSC be used in this area. The ATSC system will allow the wire height to be managed more efficiently due to the current heating, and help to reduce the loadings on the ramp structures. An additional advantage is ATSC's inherent ability to accommodate the expansion and contraction of the bridge members while maintaining pantograph security.

It is recommended that portal type structures be used to support the catenaries. These will help reduce the loadings on the existing ramp structures. The catenary poles can also be used to support the traffic and LRT signals, thereby reducing the visual clutter.

It is noted that the installation of LRV OCS over the joint use trackbed/roadway will preclude the future use of electric busses on this alignment due to differences in the power supply and collector systems.

Street running will predominantly be used in Bellevue, the Bel Red Corridor, and in downtown Redmond for several of the alternatives under consideration. In accordance with the Design Criteria, it is envisioned that a low profile simple catenary be used. This system will be auto-tensioned in order to provide the lowest wire heights commensurate with street-running operations.

It is anticipated that some form of tapered tubular pole will be used in the street running area. It is possible to coordinate pole design with urban design elements to form a blended streetscape that incorporates the other vertical elements such as street lighting, pedestrian lighting and traffic signaling into a design theme that addresses pole spacing and placement, as well as some architectural treatment. The architectural treatment can consist of decorative pole bases, paint treatments, and/or pole caps. Balance weight assemblies for the automatic tensioning system will be concealed inside of the tubular poles. In some cases, dependent on local agreements, it may be possible to incorporate street and pedestrian lighting into the OCS poles.

Dependent on the final alignment selected there are a number of options available for pole placement. They can be in the center of the tracks, or on the sidewalks. If sidewalk locations are chosen, it is probable that a span wire construction will be used instead of cantilevers to support the catenary. Span wires have less visual impact, and can support catenaries in the middle of the street area from the sidewalk locations.

The use of single contact wire in the street running areas can also be considered due to the lower operating speeds of the LRVs. It is noted, however, that this option would require extensive underground ductbanks under the sidewalks, which is already a crowded area due to utilities, much closer pole spacing, and frequent feeder risers from the ductbanks to feed the OCS. At this time this option is not recommended due to the reasons noted above, as well as the significant cost and construction impacts.

3.3 Tunnel OCS

Sound Transit's Design Criteria currently calls for fixed termination, low profile OCS to be used in the tunnels. Currently there are two tunnels requiring OCS, Mount Baker and

Mercer Island. There are also tunnel options under consideration for the downtown Bellevue area.

With regard to the Mt Baker and Mercer Island tunnels, it is recommended that a low profile AT system be considered (see Figure 3-3). Both tunnels are relatively short, (approximately 3,500 and 2,900 ft long respectively) and can be accommodated by use of a single tension length of equipment. AT equipment will be in use on both sides of the tunnels, and is, in fact necessary to handle the movement of the floating bridge on I-90, which is directly adjacent to the Mercer Island Tunnel. Due to the limited clearance in the tunnels (approximately 18 ft), AT equipment will also be necessary to effectively maintain reasonable wire heights. The use of AT equipment will also reduce the loadings imposed by the OCS supports on the tunnel structure.

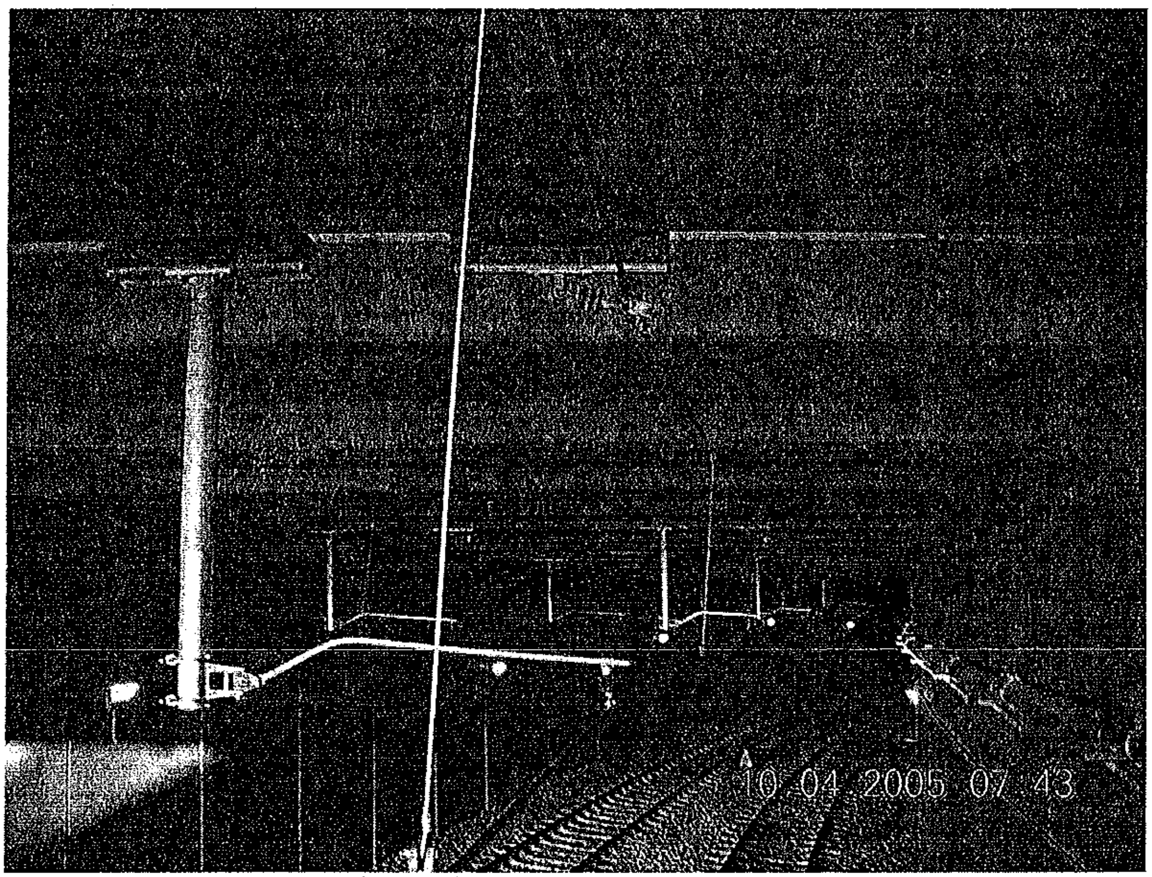


Figure 3-3 Simple AT Catenary in Tunnel

The Mercer Island Tunnel has a plenum above the roadway. The plenum is expected to remain in place and potentially has sufficient capacity to support the OCS. If not, the supports may have to penetrate the plenum for attachment to the tunnel roof. With the addition of the center wall in the tunnel, loads on the plenum are reduced from the present condition. This will be an item to be resolved during final design.

In the Mt. Baker Tunnel the OCS will be attached to the tunnel ceiling.

The type of OCS to be used in the Bellevue Tunnel Alternatives will be dependent on final design of the tunnels. Given the philosophy outlined above for the Mt. Baker and Mercer Island Tunnels, it is probable that a low profile AT system will also be considered for this application.

In all tunnels, emergency blue light stations will be installed in accordance with the relevant fire/life safety requirements.

3.4 I-90 Floating Bridge

The I-90 floating bridge is a unique structure for the installation of LRT. The structural aspects have been addressed in separate reports. The challenge of the OCS design is to accommodate the horizontal and vertical movements on the transition spans (see Figures 3-4a through c) while maintaining adequate current collection capabilities.

At the transition spans the bridge is subject to dynamic movement created by the fluctuation of the lake level, as well as by wind acting on the structure. This is complicated somewhat by the change in grade of the track profile in order to transition from an elevated approach on land to the lower level of the floating bridge itself.

In order to accommodate this movement, an automatically tensioned overlap will be installed at the transition. The overlap will be a special design with an elongated common running section to provide smooth pantograph continuity as the bridge moves. In addition, support structure spacing will be reduced to accommodate the varying change in gradient as the spans move relative to each other to accommodate the change in lake level. This solution has successfully been applied to long span suspension bridges.

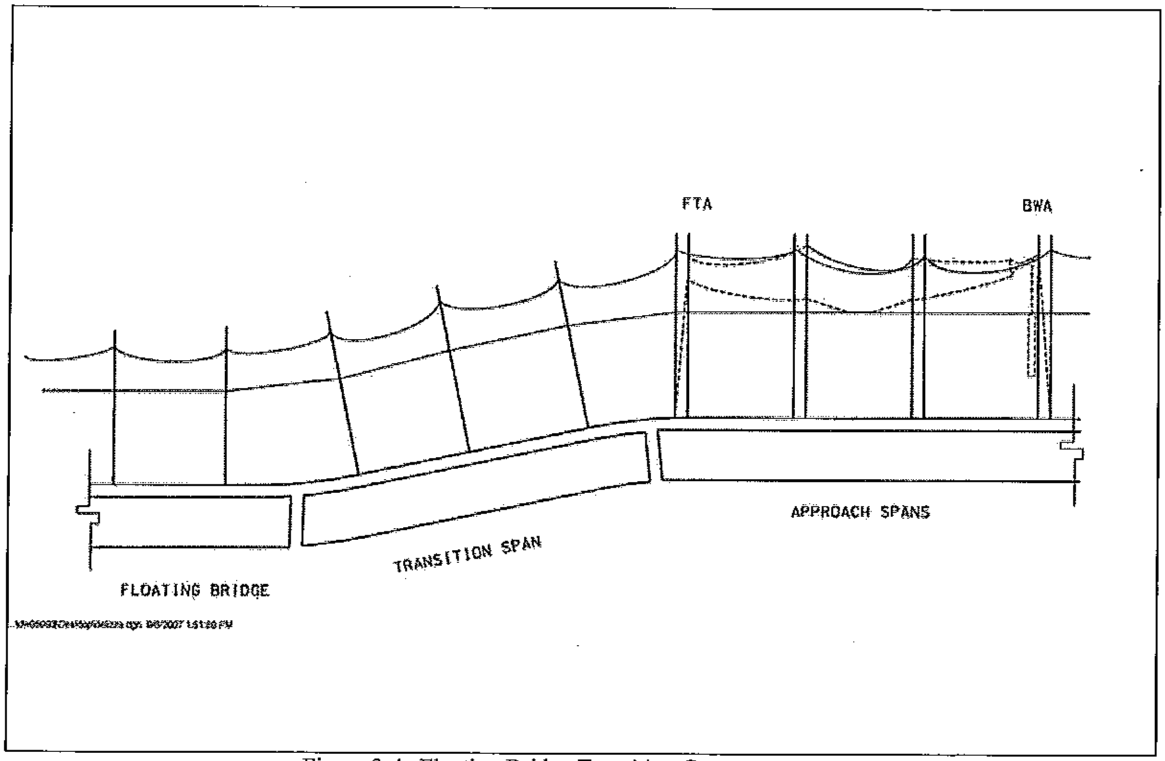


Figure 3-4a Floating Bridge Transition Concept

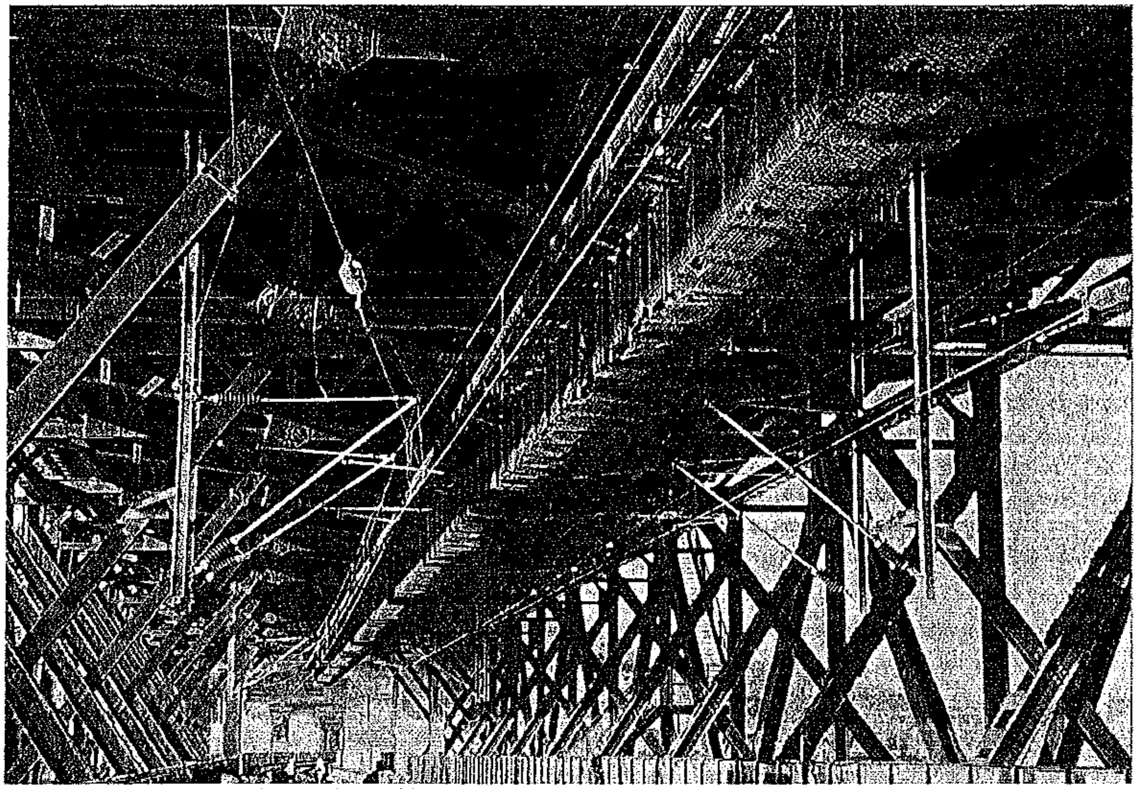


Figure 3-4b Transition overlap to accommodate bridge movement

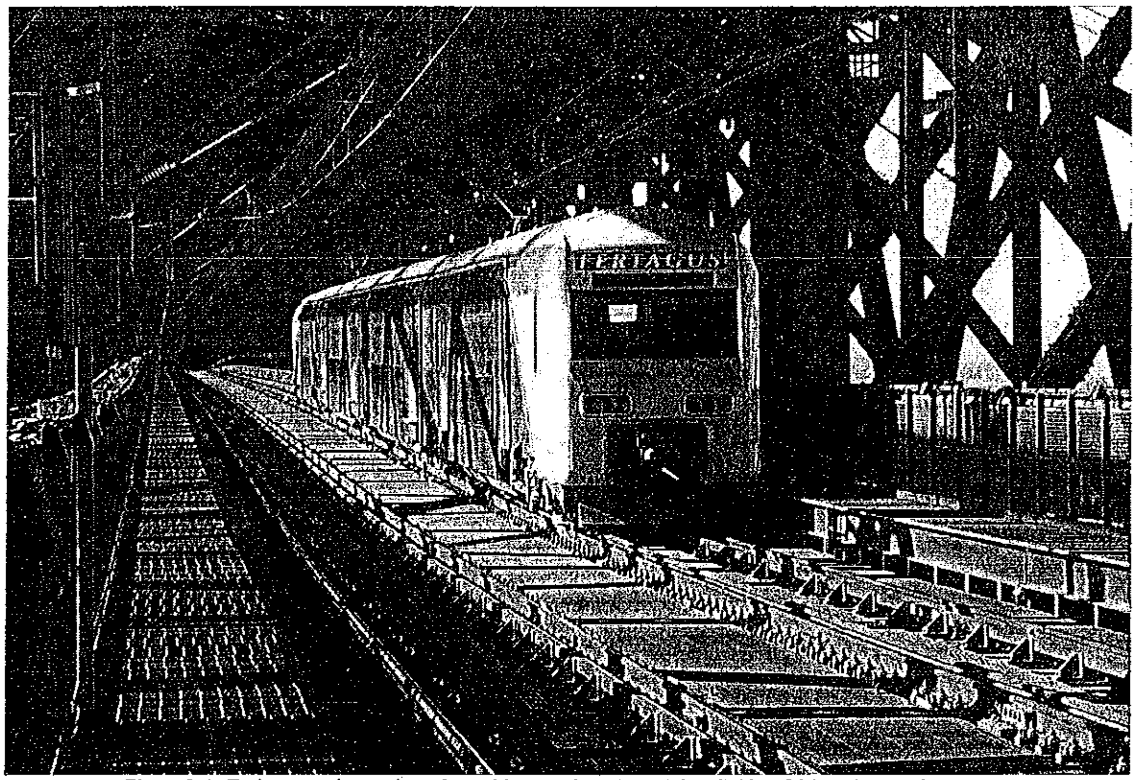


Figure 3-4c Train traversing region of transition overlap, Tagus River Bridge, Lisbon, Portugal

In order to reduce the moments that single catenary poles would apply to the deck structure moment resistant portal type structures are recommended for use on the floating section (see figure 3-5). This will also simplify the attachment to the concrete deck. The portal legs will be located to avoid interference with the WSDOT maintenance lane and the access hatches to the inside of the pontoon structure.

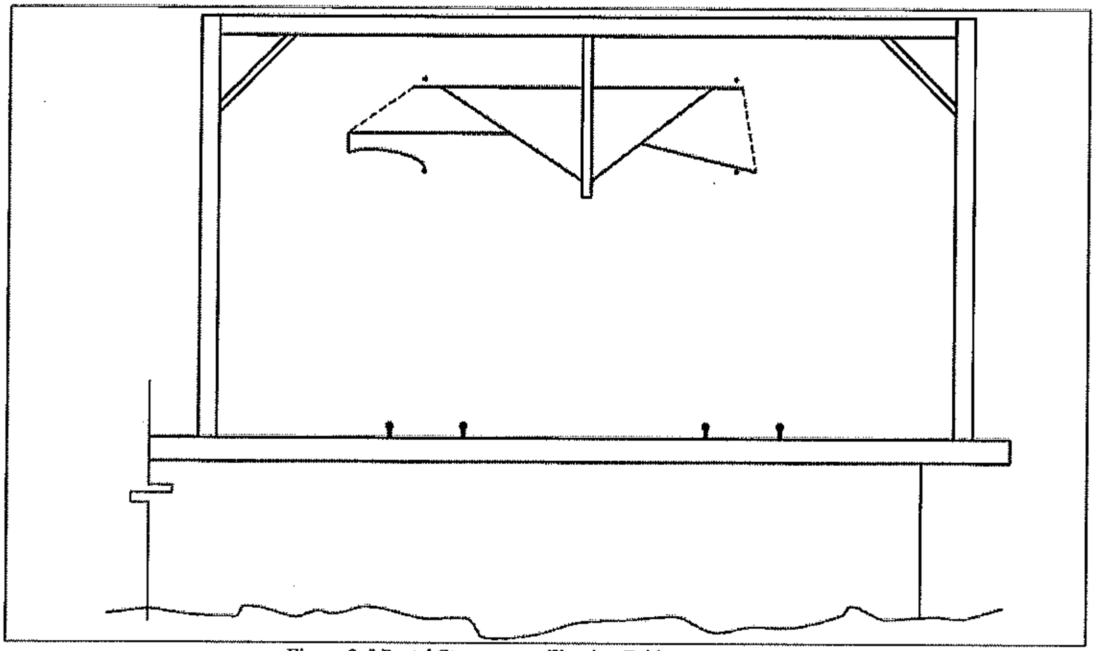


Figure 3-5 Portal Structure on Floating Bridge

Concepts for the attachment of the poles to the deck will be developed during the preliminary engineering phase, when the loadings on the poles can be fully developed. Attachment to the deck will require a detailed mapping of the post-tensioning and reinforcement in the deck. Placement of the poles and clearances will be also evaluated during preliminary engineering to optimize the transverse position of the pole. It may also require an a-typical offset base plate.

Since the track will have a sliding joint type of connection on the transition spans, negative return jumpers (see Figure 3-6) will be installed to provide continuity of the traction return circuit. These will be coordinated with the signals design to ensure ad equate track circuit protection.

3.5 Maintenance Facility

A new maintenance facility will be constructed to serve the route. The final location and configuration have yet to be determined. The maintenance facility will consist of running and storage tracks, as well as a service building.

In accordance with Sound Transit's Design Criteria, a single wire (tramway) fixed termination type OCS will be used to wire all tracks after the lead-ins from the main line. It is recommended that tapered tubular poles be used for the yard area because they are able to handle loadings along several axis much more efficiently than wide flange sections.

The contact wire will extend into the service building, and will be electrically isolated from the rest of the yard. Interlocked disconnect and grounding switches will be used to protect service personnel. The rails in the service building will be grounded to the building steel in order to avoid shock hazards. It is normally recommended that a separate substation be used for the shop building, but this will need to be further discussed during final design before a final recommendation can be made.

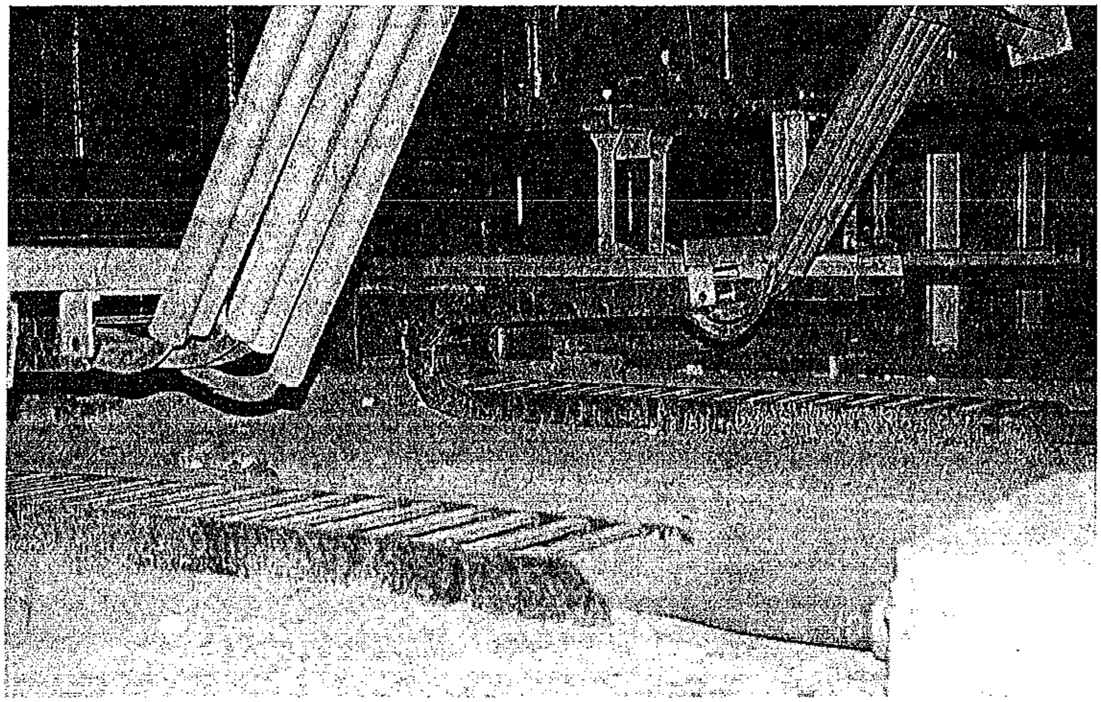


Figure 3-6 Negative return system on bridge transition

The yard OCS will be electrically isolated from the main line, as will the rail return system. It is recommended that a substation be provided to power the yard independently of the main line. There are several reasons for this, including better stray current control, and the relatively high layover loads imposed by the vehicles in overnight or mid-day storage.

4.0 Conclusions

The application of Sound Transit's existing style of OCS systems is a sound approach to take for the East Link Project. The electrical characteristics of the system are compatible with the future service, the mechanical characteristics will accommodate the climatic and alignment requirements, and maintenance inventory will be simplified.

A deviation from the Design Criteria's use of a Low Profile Fixed Termination style of OCS is recommended in the tunnels. The recommendation of this report is to consider the use of a low profile Automatic Tension system for compatibility with the catenaries on the approaches, and simplification of the transition to the I-90 floating bridge.

Simple Automatic Tensioned OCS can be used on the I-90 floating bridge. The overlaps at the approach spans will be designed to accommodate the span movement. Portal type structures will be used on the bridge to reduce the moments imposed on the top slab of the floating sections.

Joint use of D-2 Roadway

TO: Roger Koester/Parsons

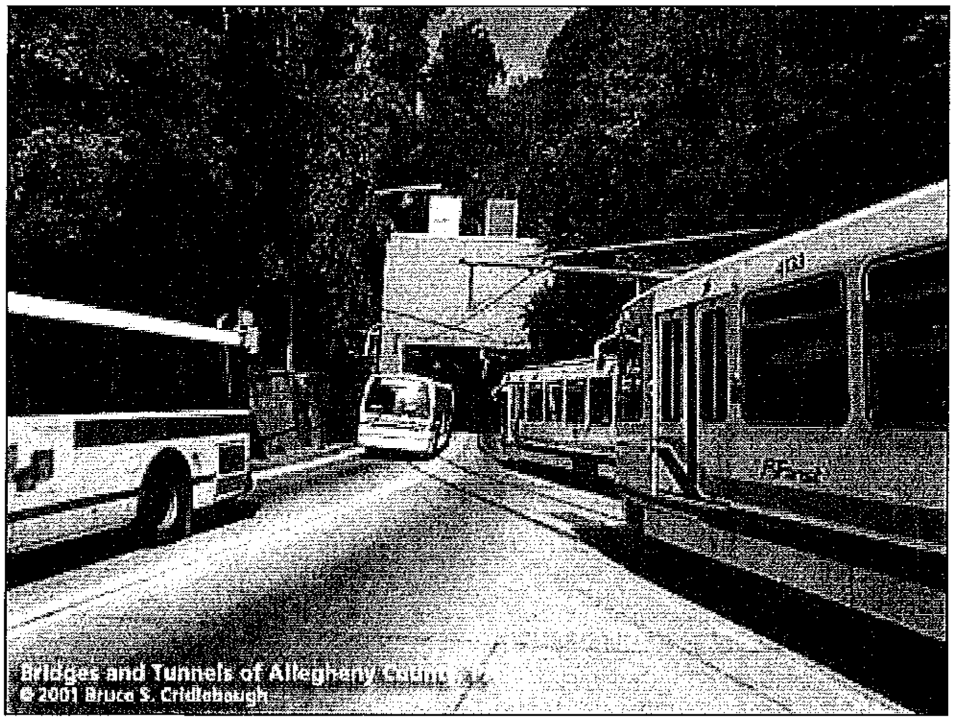
COPIES:

FROM: Winn Frank/Parsons

DATE: September 25, 2007

Sound Transit is presently considering operating about a 1.4 -mile joint use corridor for Sound Transit buses and LRT trains on a segment between Rainer Station and downtown, referred to as the D-2 Roadway. The principal concern is that of safe operation. Conceptual engineering for the Sound Transit East Link Light Rail corridor is presently underway. This review is intended for consideration regarding decisions to be made during conceptual engineering.

There are few comparable transit bus and LRT joint use segments in North America. One example is Pittsburgh's Mount Washington Transit Tunnel which is 3,400 feet in length and has a grade of nearly 7 percent. The tunnel was originally opened for trolley rail service in 1904, and has very narrow track centers. At both ends of the tunnel, the opposing routes intersect each other as shown in Figure 1 below. The LRT lines utilize two-car trains in the Tunnel, and the maximum authorized speed for both trains and buses is 25 MPH. On October 29, 1987, a PCC trolley derailed because of braking problems, resulting in a few injuries but no fatalities. The Tunnel traffic is not regulated by a signal system.



(Used with permission)
Figure 1 South Portal of Mount Washington Transit Tunnel, Pittsburgh, PA

Another joint use example is the downtown core area of Calgary, AB. This area has a 12-block section of 7th Avenue restricted to access by LRT trains and transit buses, as well as emergency vehicles. It is a free fare zone with multiple stations. This corridor uses line-ofsight operation, with a maximum authorized speed on 7th Avenue of 25 MPH. Separation of cross street traffic, trains, and buses is controlled by conventional traffic signals.

Engineering Description of the Joint Use Segment

The proposed LRT and bus joint use segment is about 1.4 miles in length. The route is mostly on an elevated structure, with eleven horizontal curves. Presently, buses and HOV's operate in opposing directions on a barrier separated roadway. Each lane is 20 feet wide. Figure 2 presents a general configuration of the D-2 Roadway and shows the assigned traffic lanes.

Maximum Authorized Speed

LRV Simulations previously performed for the joint use segment resulted in a run time of 175 seconds (0:2:55), for an average speed of 28 MPH, but maximum speed is limited to 40 mph by the Automatic Train Protection (ATP) system. It is anticipated that bus operations would be limited to 40 mph; thus, LRVs, and buses operate at a generally uniform speed range. The achievement of a uniform speed range maximizes throughput and promotes safety.

The D-2 Roadway alignment has two restricting horizontal curves indicated in Table 1 below.

Begin/PC	End/PT	Radius	Length	Max Design Speed
1001+27.73	1001+64.51	500 feet	37 feet	25 mph
1013+39.78	1017+24.57	300 feet	385 feet	20 mph

Table 1 Restricting Horizontal Curves

Headways

The planned LRT headway in peak period is 9 minutes. The bus mode headways would be 26 per hour during peak (Year 2030). Thus there would be a total of 33 vehicles per hour during the peak, or a vehicle every 90 – 110 seconds on average. Table 2 presents a listing of buses using the D-2 Roadway in 2030. Those bus routes not listed have been either re-routed, truncated, or deleted and would not use the D-2 Roadway.

	2030 Headway		2030 Buses per Hour	
Route	Westbound	Eastbound	Westbound	Eastbound
AM Peak				
212	6	15	10	4
214	12	0	5	0
214.5	30	0	2	0
216	25	0	3	0
218	10	0	6	0
Total AM Peak			26	4
PM Peak				
212	15	6	4	10
214	0	12	0	5
214.5	0	30	0	2
216	0	25	0	3
218	0	10	0	6
Total PM Peak			4	26

Table 2 Buses using the D2 Roadway

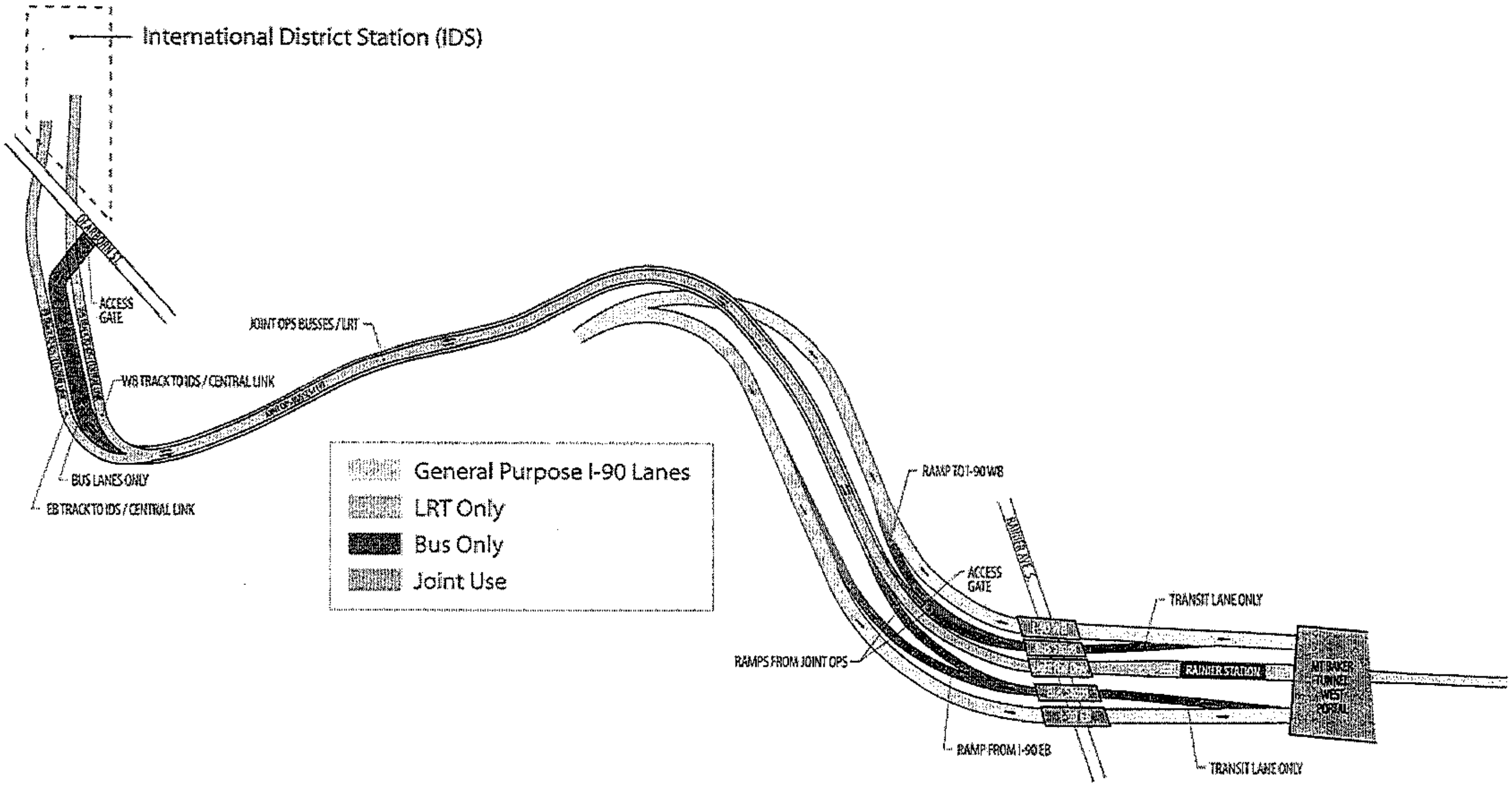


Figure 2 General configuration of the D-2 Roadway

Operating System

Consistent with the Central Line Downtown Seattle Transit Tunnel (DSTT) operations, an automatic signal and tracking system would be used on the D-2 Roadway. By the time East Link opens, Central Link will have operated for many years in a joint operations mode with buses in the DSTT. This joint operations system has the following characteristics:

- The Seattle LRT line is Automatic Train Protection (ATP) capable. The D-2 line segment must be ATP equipped at the downtown end where it begins the ATP merge with the north/south line and it will be ATP equipped crossing the bridge where it will not be restricted to trolley speeds. Therefore, there should be continuous ATP operation between these two segments. Note that DSTT will be operating jointly with higher speeds and much heavier traffic than that planned for the D-2 Roadway.

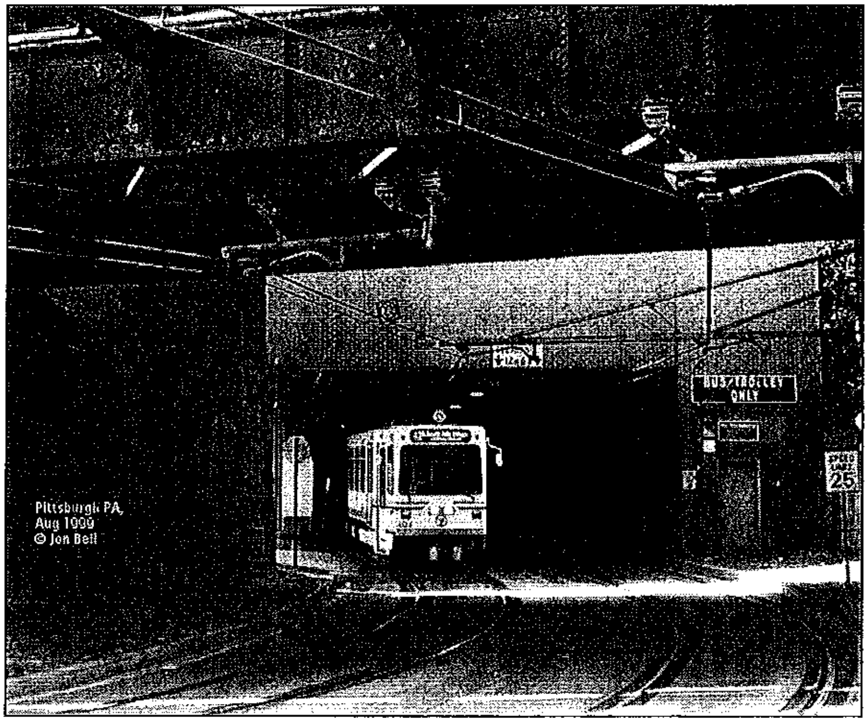
- Detection is the building block for safe operations that are not completely time separated. The rails installed in D2 will be isolated by rail boot or other means which permits track circuits. The current signaling concept is to have continuous detection of the LRV with tracking display at the Operations Control Center (OCC). Further, a check in – check out bus detection system has been successfully installed and tested in the DSTT.

- Seattle's DSTT joint operation signal system provides detection and signals for both bus and rail. Safety at the merge and linear separation of the LRT/bus modes are built into this system. In addition, the OCC has the capability to track both buses and trains that are in the joint area, their vehicle numbers, approximate locations, and the signal conditions.

Two separate types of wayside signals are envisioned for the D-2 Roadway; LRT Bar signals would govern LRV movements; color light signals would govern bus movements. Both would be interconnected and movements "managed" by an interlocked signal processor governing signal progression and providing safe vehicle separation. Vehicle identification and tracking would employ track circuits for LRVs, and automatic vehicle identification technology (tag readers) for buses. The bus tag reader system tracks the location of each bus by serial number. Drivers would be responsible to operate under signal indication. There would be automatic overspeed penalty stopping for LRVs.

The bus on-ramps to the D-2 Roadway would be equipped with gates to prevent auto/truck traffic from entering the joint roadway. These gates would be raised by the automatic vehicle recognition tags on each bus or other wayside sensor. At the west end of the joint operating segment (the three way intersection of Airport Way, Dearborn Street, and 5th Avenue) signal/gate activation design, and or gate location, would ensure that a bus entering the joint use segment would not be stopped blocking that busy intersection. This may require integration with the City's traffic signals control system.

A signal indication would regulate the insertion of buses between LRV movements onto the joint use roadway. Figure 3 is an example of this integration at Mount Washington Tunnel in Pittsburg, PA.



(Used with permission)

Figure 3 Mount Washington Tunnel – Pittsburgh, PA. Note signal, Bus/Trolley only sign, and speed limit sign

Buses exiting the joint use segment at either the east or west end would not intersect an LRT moving in the opposing direction. In the event that signal circuitry is located in the area of an LRT station, motion detection and constant time capabilities could be incorporated to minimize bus waiting/insertion time when an LRV is stopped at the station.

Safety Training

It is possible that the DSTT joint operation would continue during D-2 Roadway operations. If this is the case, minimal new operator training would be required. Special instructions for the D-2 Roadway, including a 40 MPH maximum authorized speed and the 25 -20 MPH restricted curves should be included in the Sound Transit Operating Rulebook or Operating Procedures.

D-2 Roadway design

Initial engineering indicates that the existing HOV structure cannot support the additional weight of the LRTs' embedded track without removal of the median barrier. See Figure 4, next page. The safe and successful integrated LRT and bus operations at Pittsburg, with no separation barrier, and under more difficult physical conditions than is contemplated in D-2 segment is noted herein. With the precedent set by Pittsburg and Calgary it is suggested that a separation barrier is not necessary for the joint LRT/bus operations on the D-2 segment.

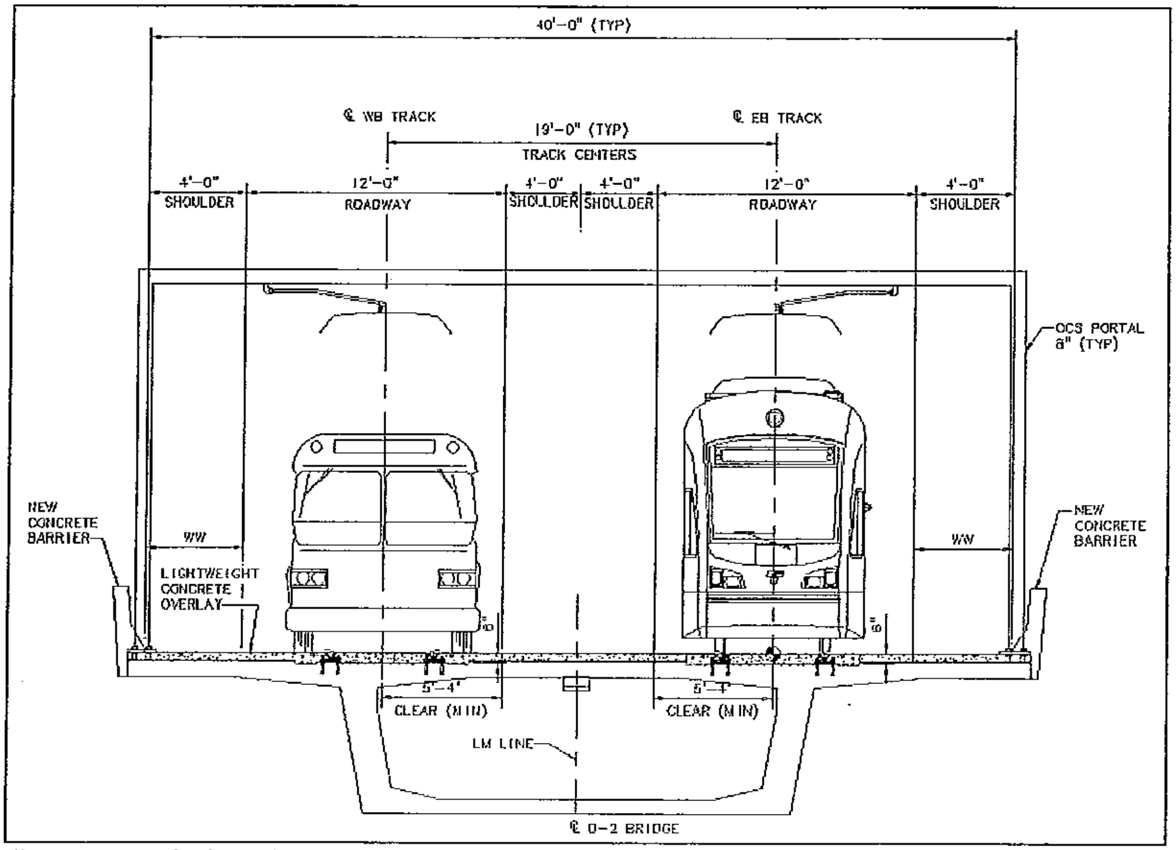


Figure 4 Typical Section - Bridge D-2 Joint Operations
LRT Double Track - Embedded Girder Rail - with Express Buses

Bus Integration with a 4 minute LRV Headway

Simulations were run to determine the number of buses that would be able to run between LRVs operating on four minute headways. It was found that the maximum number of buses would be three; but this would be a very tight schedule, a more practical number would be two buses. Figure 5 indicates the timings for the three bus intervals. These timings are based on the computation of safe braking distances for a typical bus and LRV. The assumed braking rate, (on level ground) for the LRV is 2.00 MphPs and for the bus is 9.45 MphPs. These rates were adjusted for delayed reaction time (8 seconds), inefficient brake performance (-35%), and by using the Long Island RR "Effective Feet" (EF) methodology to account for gradient and reaction time. For example, for the westward (to IDS) downhill grades, the effective distance for braking increased an average of 30.0% as the grade continues to accelerate the train against the braking force; resulting in a braking time of 1:30. For the eastward (to Rainier) uphill grade decreases the effective distance for braking by -5.2% as the adverse grade helps to slow the braking train. These rates are based on a maximum authorized speed of 40 mph for LRVs and buses on the D-2 Roadway, and the safe braking distance of each bus or LRV required to stop from 40 mph, clear of a vehicle stopped ahead of it.

It is evident that the minimum headways eastbound can be much closer than those west bound because the adverse grade helps trains stop. However, if overall LRV schedule symmetry is desirable, the longer braking distance required for the west bound movement determines the planning headway. This assumption was used to develop the timings shown in Figure 5.

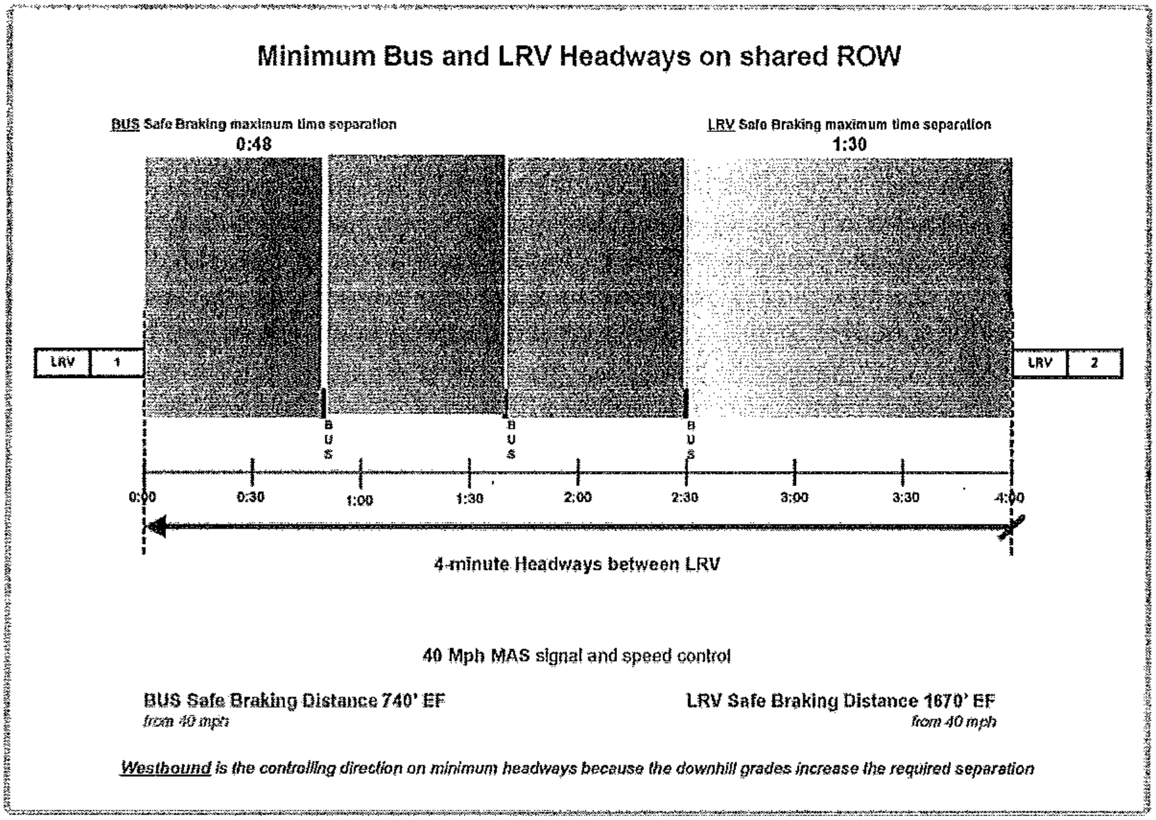


Figure 5 Minimum Bus and LRV Headways

Further, it should be noted that with the three bus scenario, the bus schedules will not be symmetrical; that is, the buses will be bunched 1-2-3 on 48 second headways, followed by a 1:30 space for the LRV, then a repeat 1-2-3 spacing. With two buses, the schedule is somewhat more symmetrical with a 1-2 spacing on 1:12 headways

Snow and Ice Conditions

According to the King County Hazard Mitigation Plan, Seattle averages one or two snow storms with appreciable accumulations during a winter season. The average snowfall in December is 1.8 inches and in January 1.4 inches. In November, February, and March, the average snowfall is less than an inch. However, every few years there occurs a single snowstorm which far exceeds the monthly average. There is little snow removal equipment or budget associated with extreme conditions. Consequently, heavy snow conditions have resulted in loss of electric power and transportation restrictions.

The Sound Transit operations staff will monitor weather conditions and maintain contact with the weather bureau during those times when snow has been predicted. The most effective way to keep the rail flangeway and contact wire clear of modest amounts of snow or ice is continuous LRT operation on the line, particularly during night time periods. Buses may be equipped with chains and operate on schedule. Operating with chains should not damage imbedded rail. The spreading of salt and sand (if such truck equipment is available) may be considered when snow accumulations of 1-2 inches or more are predicted.

On rare occasions, snow accumulation could reach a point where plowing becomes necessary. For the most part, the need for plowing is an administrative determination based on the weather forecast, current/predicted temperatures, and current operating conditions as reported from the operators of the LRVs and buses. Should plowing be judged necessary, the snow should be plowed to the center of the Roadway where there is an 8 foot shoulder. Although some operational flexibility would be lost because passing a disabled vehicle may become impossible, emergency egress for bus passengers is maintained. No special provisions are necessary for snow plows to operate on an embedded track structure. In the extreme case that plowed snow accumulates and extends into the road way, LRV and Bus operations should stop and snow removal operations begin (Front end loader and dump truck).

Special operating procedures may be required in high gradient track segments. For example, snow removal crews may have to be dispatched to those locations as wheel slip may prevent LRV operations during heavy snow conditions.

EAST LINK PROJECT, PHASE 2
Discipline Review Comment Form - Form DRC

Comments

Reviewer	Originator's Initials/ Organization:	Package	Date	Comment No.	Reference/ Subject*	Review Comment	Initial Code**	Comment Assigned to	Response	Response By	Final Code***	Open/ Closed (Date)
Wash. State Dept. of Transportation	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	1	Structural Capacity	There is a brief comment on page 5 (D-2 Roadway Design) which states: "Initial engineering indicates that the existing HOV structure cannot support .the additional weight of the LRT's embedded track without removal of the median barrier." WSDOT will require a more extensive analysis of the D-2 roadway - structural/seismic prior to approving the conversion of this structure to LRT and/or joint LRT/Bus usage. %		Koester, Roger	Purpose of report was to evaluated operational aspects of a joint bus/LRT operation. The cite comment was derived from an earlier WSDOT report prepared by KPFF. Structural analysis of the bridge will be completed in the next phase.	Koester, Roger		
Wash. State Dept. of Transportation	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	2	Bikepath Extension D-2 Roadway	Shouldn't the D-2 analysis also address the potential extension of the bikepath from the Mount Baker Tunnels along the D-2 roadway? (explain why it wont work, ST to decide ....)		Koester, Roger	A bikepath extension on the D2 would restrict the joint use operation to much narrower Guideway/Busway and result in limited and potentially unsafe operation. As presently configured there is no safe means of exiting the D2 bridge at Dearborn Street. there also does not appear to be easy way to connect the bikepath from the tunnel exit at 23 Street to the center roadway at Rainier.	Koester, Roger		
Wash. State Dept. of Transportation	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	3	Figure 4, page 6	The pole connection shown on this detail (mounted to the bridge deck) may not be acceptable to WSDOT. This comment has been made on a number of occasions to Sound Transit. It is recommended that ST consider sonie pre design engineering to ensure that this connection will work. At this time, WSDOT can't approve this type of connection to the existing structure.		Koester, Roger	For the anticipated portal system the connection will carry vertical loads, shear loads and minimal bending moments. Analysis of the loads and connection will be completed in the subsequent phases	Koester, Roger		
Wash. State Dept. of Transportation	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	4		The purpose of this report isn't partic—larclyle ar. There is no executive summary or recommendations contained in the report. Similar to the two earlier reports, the information contained in this document is relatively generic and provides little for WSDOT to review/approve		Koester, Roger	The purpose of the study is to assess the operational requirements for a joint LRT and bus use. This report addresses the general control systems, frequency and timing to bus interphasing with the LRT and safety of the operation.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	1	Policy Issues around Joint Use of D-2 Roadway	E-3 versus D-2		Koester, Roger	The D2 teminology has been used generally to describe the area between the IDS and Rainier along I-90. The WSDOT KPFF report also used this name. The E3 is the bus connector to I-5	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	2	Policy Issues around Joint Use of D-2	Who owns the joint use decision - ST or WSDOT?		Koester, Roger	ST is in ongoing discussion on the use of the I-90 facilities. This will part of an overall agreement.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	3	Policy Issues around Joint Use of D-2 Roadway	Ownership and O&M of D-2		Koester, Roger	ST is in ongoing discussion on the use of the I-90 facilities. This will part of an overall agreement.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	4		Within the title and introductory paragraph, insert "I-90" in front of D-2 Roadway, so people know what we're talking about.		Koester, Roger	A figure is included to show the area of interest.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	5		It's not clear if wait time for buses at gated entrances has been factored into the safe headway estimate. A bus will need to stop, be cleared, and wait for gate to rise before entering into the roadway.		Koester, Roger	Extra wait time for the bus will need to evaluated as part of the overall operations and considered in the analysis of schedule impacts of Joint Use versus the street running operation. This report analyzes the operational aspects of a joint use operation and does not address merits of joint use versus street running.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	6		Current operating speed for buses on the D-2 is less than 40 mph due in par to the horizontal curvature. Vertical curvature also has an effect. I'm concerned with the _ presumption on page 2 that bus and light rail speeds would be as uniform as they are on arterial streets		Koester, Roger	The LRT vehicle and the bus will have similar geometrically constrained speeds. The operation will also have some time separation between the vehicles.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	7		Does joint use of the D-2 roadway also mean bus use of the Rainier transit stops? This appears to be the case in Figure 2. How do passengers move between the different transit systems - down to Rainier and back up?		Koester, Roger	Access from the existing flyer bus stop at Rainer will be via the access ramps from Rainier. The Rainier Station will have a ramp connecting the existing ramp system under the Rainier Bridge.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	8	Page 4	it is stated that the DSTT will operate jointly with higher speeds than D-2.. I recall certain pieces of DSTT having 20 mph speed limits if not lower. What is the bus operating speed in the tunnel now that it's reopened?		Koester, Roger	KCM has it signed for 20 although the design speed and signage speed was intended to be 25	Gedosch, Mark		

Reviewer	Originator's Initials/ Organization:	Package	Date	Comment No.	Reference/ Subject*	Review Comment	Initial Code**	Comment Assigned to	Response	Response By	Final Code***	Open/ Closed (Date)
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	9	page 6,	for clarity please identify what time period the number of buses is within. It appears that it is two buses within the four minute LRV headway. And what is the number of buses that could be accon-modatedw ithin the initial LRV headway (it is not four minutes year of opening)		Koester, Roger	Figure 5 shows the bus spacing for a four minute headway situation. Longe periods would accommodate proportionally more buses.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	10		There should be a brief discussion of how joint use does or does not affect response to vehicle breakdowns (bus or LRV) and emergencies.		Koester, Roger	Operating protocols will need to be developed in the case of a disabled vehicle.	Koester, Roger		
Barley, Mark	WSDOT	Joint Use of D-2 Roadway Memorandum	10/17/2007	11		Joint use of D-2 would appear to prohibit the idea of connecting the E-3 busway up to 5th & Airport, an idea that I've heard in conjunction with the future of DSTT being LRT only. If this may be true, it should be identified as a question to be investigated.		Koester, Roger	If there is a future intent to connect the E3 busway traffic south of the DSTT to the 5th & Airport roadway fitting ramps would be problematic, particularly on the western side of the D-2 roadway given the restricted available space between existing building structures and the East Link ramps into the DSTT. This needs to be studied further during Preliminary Engineering. The D2 roadway is independent of the E3 busway. Joint use will not interfere with E3 use until LRT operations increase to a level that makes continues joint use of the DSTT problematic.	Koester, Roger/Kambol, Steve		
Sound Transit	Sound Transit	Joint Use of D-2 Roadway Memorandum	8/29/2007	1		1. The memo still does not describe sufficiently the precedent being set by the DSTT joint operations. After the first sentence of the second paragraph, please add the following language or something comparable: The precedent of most importance here is the joint operations Downtown Seattle Transit Tunnel (DSTT) and its signal system. By the time East Link opens, Central Link will have operated for many years in a joint operations mode with buses in the DSTT. This joint operations system has the following characteristics: • The Seattle LRT line is Automatic Train Protection (ATP) capable. The D2 line segment must be ATP equipped at the downtown end where it begins the ATP merge with the north south line and it will be ATP equipped crossing the bridge where we will not be restricted to trolley speeds. Therefore, there should be continuous ATP operation between these two segments. Note that we will be operating jointly with higher speeds and much heavier traffic in the DSTT. • Detection is the building block for safe operations that are not completely time separated. The rails installed in D2 will be isolated by rail boot or other • Seattle's DSTT joint operation signal system provides detection and signals for both bus and rail. Safety at the merge and linear separation of the LRT/bus modes are built into this system. In addition, the OCC has the capability to track both buses and trains that are in the joint area, their vehicle numbers, approximate locations, and the signal conditions. Although this signal system cost will be more than a stop light, it is insignificant when the benefits of superior safety and higher speed are considered.		Koester, Roger	Agreed, description incorporated in the report	Koester, Roger		
Sound Transit	Sound Transit	Joint Use of D-2 Roadway Memorandum	8/29/2007	2		2. Memo should discuss whether joint operations would be feasible at the ultimate East Link frequency of a 4-minute rail headway. If not, what is the minimum headway at which joint operations can continue? We will want to compare the duration of potential joint operations against the incremental cost. Note: this comment was sent previously		Koester, Roger	Agreed. This discussion has been included.	Koester, Roger		
Sound Transit	Sound Transit	Joint Use of D-2 Roadway Memorandum	8/29/2007	3	3. Figure 2	At the entrance to IDS the graphic has EB and WB reversed.		Koester, Roger	Agreed. Figure has been changed.	Koester, Roger		